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Soil-Cement and Soil-Lime Stabilization of Weathered Surface Clays in Southwestern Ontario

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Soil-Cement and Soil-Lime Stabilization of Weathered Surface Clays in Southwestern Ontario

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Preface

The research presented in this report deals with the effectiveness of cement and lime as stabilizers for the weathered, surficial, clayey soils of southwestern Ontario. The work follows logically our previous Project L-3 which dealt with the compaction-strength behaviour of natural (unstabilized) soils from different depths in the weathering profile.

Project L-3, completed in January 1975 [12], clearly established that the weathered surface soils compact much less efficiently than the deeper soils due to the presence of abundant smectite (swelling clay) produced by oxidation of the original soil chlorites. The specific purpose of this project has been to establish the stabilization characteristics of the soils using cement and hydrated lime as stabilizers.

The work was commenced during the summer of 1975 and originally scheduled for completion by August 31, 1976. This latter date was extended to November 30, 1976 to enable completion of the concurrent M.E.Sc. thesis by L. DiNardo who performed most of the testing for this project. The work has been supported under the Ontario Joint Transportation and Communications Research Programme and is known as Project L-6. The work has been carried out in the Soil Mechanics and Soil Chemistry laboratories of the Faculty of Engineering Science, The University of Western Ontario. Support facilities in the Materials Science Laboratory were used extensively for the X-ray studies.

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1/ Introduction

The clay deposits in southwestern Ontario have a surface area in excess of 6475 km^2 (2500 sq. mi.) and are thus of considerable importance to highway engineering, especially since supplies of granular base course materials are rapidly diminishing.

The clayey soils consist of both lacustrine silty clays and "waterlain" tills and are thus either normally consolidated or only slightly over-consolidated at depth. At surface, they are characterized by a stiff to hard, brown, oxidized, partially leached, fissured crust that varies from 2 to 5 m in thickness. The crustal clayey soils are extensively used as borrow for construction of embankments and highway sub-base. Unfortunately, the mineralogy, especially the content of swelling clay, varies greatly depending on the depth of excavation, thus greatly affecting the engineering properties of the borrow materials.

The compaction-strength characteristics of the crustal soils from Sarnia, Glencoe and Warwick were clearly established as a function of mineralogy in Project L-3 [12], completed on January 31, 1975. The purpose of this Project (L-6) has been to establish the effectiveness of both cement and lime as stabilizers for the crustal soils. Early in the project, the soils at Glencoe and Sarnia were resampled for the stabilization studies, however, the scope of the testing expanded so enormously because of the many variables involved, that only the Sarnia soils were actually tested.

Emphasis has been placed on the interrelationship of reaction chemistry and the physical properties obtained through lime and cement stabilization. In particular, the importance of the swelling clays and any amorphous constituents in the soils has received special attention, since these factors would markedly influence the time rate of strength gain, the ultimate strength, and also the effectiveness of trace additives.

In the first part of this report, the significant engineering test results and recommendations are presented in summary form. Appendices at the rear of the report contain details of the literature review, testing procedures, clay mineralogy, etc.

2/ Scope of Project and Procedures

2.1/ Scope

Soil sampling comprised the first phase of the program followed by selected mineralogical and compaction testing to ensure that the results from the new soil samples would be similar and comparable to the previous results from Sarnia. Once the entire profile was checked, extensive stabilization testing was commenced on bulk samples from 0.3 and 2.4 m depth.

The compaction tests were run on soils stabilized with 0, 2, 4, 8 and 12% of cement and of lime. Unconfined compression tests were run on samples of the above, cured for 7, 28, 60 and often 90 days. Swelling tests were run on samples both uncured and cured to assess the efficiency of stabilization with curing time and content of stabilizer.

To assess the importance of clay mineral reaction with the cement and lime stabilizers in the creation of strong cementation bonds, enough of the < 2 μm fraction was obtained from the 0.3 and 2.4 m bulk samples to conduct both compaction and strength-time experiments.

The last phase of the project was to extract the amorphous constituents from the < 2 μm soils prior to stabilization to assess the importance of this soil component on strength gain.

Completion of the project required a total of about 800 separate engineering tests to be run during the course of about twelve months.

Throughout the project, mineralogical and chemical controls were maintained as well as possible, using such tests as X-ray diffraction, free moisture determination, free moisture salinity, pH and a token amount of electron microscopy.

2.2/ Sampling and Storage

The weathered crust at the sampling site was excavated by backhoe to a depth of 2.5 m and a continuous channel sample taken for mineralogical and index testing. Two bulk samples were taken at 0.3 and 2.4 m depth for the compaction-stabilization test program. All samples were immediately placed in plastic bags or glass jars to prevent air-drying.

All of the soil within the testing depth was of hard consistency having an *in situ* shear strength in excess of 215 kN/m^2 which was the practical limit of pocket penetrometer used. All sampling was completed in a two day period during the fall of 1975. The bulk samples were dried slightly, thoroughly mixed mechanically and stored as stock samples in large plastic buckets at a moisture content of about 10%. Pebbles greater than 1 cm in size were removed manually at this stage.

Prior to compaction, the above stock samples were passed through a 4 mesh sieve and only the < 4 mesh fraction was used for compaction.

Separation of the < 2 μm fraction for special testing was done by dispersing the soil in distilled water and collecting the < 2 μm suspension after a suitable sedimentation time. The soils were collected by flocculation by dilute $CaCl_2$. The clay fraction was then air-dried for storage. Details of the fractionation procedures may be found in Appendix B.

2.3/ Compaction Testing

In all cases, the < 4 mesh soil used for compaction (called "whole soil" throughout this report) was a never dried, damp sample (8 to 10% moisture). All testing on the < 2 μm fraction was carried out on air dried powder which had been passed through a 100 mesh sieve. Mixing was done manually with a spoon and the percentage of water and stabilizer was determined as a weight percentage of the dry soil. In the case of soil plus stabilizer, the percentage of water is referred to the weight of the total dry mixture (soil plus stabilizer).

The mixing time in all instances of soil plus stabilizer was less than 10 min, a factor especially important for cement mixtures in which early reaction tended to be a difficult variable.

Compaction was carried out in a Harvard Miniature compaction mould using a tamper having a spring which yielded a pressure of 1170 kN/m^2 at the foot of the tamping tool. Each specimen was compacted in 5 layers with 20 tamps per layer and when extruded from the mould was 71.5 mm in height by 33.3 mm diameter. Compaction by this method is known as kneading compaction and is somewhat similar to field compaction by sheep's-foot roller.

Specimens for strength testing were compacted statically to values of maximum dry density determined by the above-described kneading compaction procedures. The hydraulically-applied static loads were kept on the specimen for 30 min. Upon extrusion, the specimens were weighed, wrapped in plastic, waxed and stored in a curing box for later strength testing.

2.4/ Unconfined Compression and Swelling Tests

Unconfined compression testing was carried out at a strain rate of 0.5%/min with a load cell reading taken at each 0.13 mm of axial deformation. Sufficient points were obtained to plot stress-strain curves for all samples.

Swelling tests were carried out only on specimens compacted to maximum dry density in a split Harvard Miniature mould from which a central ring of soil, 0.84 cm in height, could be extracted. A static normal load of 6.895 kN/m^2 (1 p.s.i.) was applied to all samples through porous stones against each face prior to addition of distilled water. The tests were carried out at 18°C in a constant temperature environmental room and normally took 25 days to complete.

2.5/ X-ray Diffraction

X-ray powder patterns were run on a General Electric XRD-5 diffractometer using Cu Kα radiation generated at 50 kV and 15 mA. Specimens of < 2 μm fraction were centrifuge oriented onto ceramic plates and X-rayed in an oriented state to enhance the clay mineral peaks. Details of the X-ray procedures may be found in Appendix C.

2.6/ Chemical Testing

Determinations of the soil pH, with and without stabilizers, were done on air-dried samples to which distilled water had been added to yield a soil:water ratio of 1:2.

Salinity determinations were obtained by conductivity measurement of samples using a soil:water ratio of 1:5.

The amorphous component of the < 2 μm soil fractions was removed by sequential extraction by concentrated HCl and NaOH solutions until a constant time rate of extraction was achieved. The extracted soil was then subjected to compaction, stabilization and strength testing.

Carbonate analyses were run by gasometric methods using the procedure developed by Dreimanis, 1962 [2].

Details of the chemical and mineralogical test procedures employed may be found in Appendix C along with a discussion of pertinent problems and the methods used to solve them.

3/ Site Geology and Mineralogy

3.1/ Site Location

The soils investigated are located 6.5 km southeast of Sarnia, Ontario, at the Blackwell Road landfill site as shown in Figure 1. The area is characterized by flat grassland mainly used for farming with occasional undeveloped forest areas. The site is the same as that sampled for Project L-3 so continuity of research findings could be ensured.

3.2/ Geology and Soil Conditions (General)

The physiographic region encompassing the site is known as the St. Clair Clay Plain which covers an area of about 5800 km^2 in southwestern Ontario. The sedimentary deposits consist of normally consolidated, waterlain tills of late Wisconsin age.

The most extensive published study of these deposits is that by Ogunbadejo, 1976 and Quigley [13], who divided the section into upper and lower till sheets on the basis of detailed carbonate analyses plus many other physicochemical and engineering indicators as shown in Figures 2 and 3.

The soil conditions at the Blackwell Road site, as shown in Figure 2, indicate the presence of a desiccated and oxidized crust having a total thickness of close to 7 m. Within this crust, the soils are clayey (35 to 40% clay size) and of stiff to hard consistency. Within the crust, the natural moisture content is about 15 to 20% which is generally close to the plastic limit. The liquid limit increases towards surface from values of about 38 to 52% + just below the topsoil. This increase in liquid limit indicates the presence of an increasing proportion of smectite (swelling clay) towards the surface; a feature also indicated by the increasing glycol retentions and cation exchange capacities in Figure 3.

An additional feature of the regional soil conditions is a downward gradient discussed in some detail by Goodall, 1974 [5]. This downward gradient ensures a low groundwater level during the summer and fall and is responsible for the extensive oxidation discussed later.

3.3/ Geology and Soil Conditions (Sampling Trench)

Details of the soil profile exposed in the 2.5 m sampling trench dug for this investigation are given in Figures 4 to 6 inclusive. Two photographs showing the grass plain on which the trench was dug and the nature of the trench itself are given in Figures 4a and 4b. The soil consists of highly fissured, generally brown clayey silt or silty clay with scattered pebbles. About 15 cm of topsoil caps the section.

Carbonate depletion from the upper 0.3 m of the profile is clearly shown in Figure 5, however, significant carbonate (~40%) is present in the sample from 0.3 m depth. The calcite:dolomite ratio is about 0.7 which is characteristic of the upper till as shown in Figure 3.

The natural moisture content (Figure 6) increases towards the surface from 12.5% at 2.2 m depth to 20.5% at 0.2 m depth. This represents the profile characteristic of a wet September (1975) and is somewhat wetter than the summer profile shown in Figure 2.

The liquid limit increases from 21.5% at 2.2 m depth to 51.5% at 0.2 m depth reflecting a marked increase in activity from 0.4 at depth to 0.7 near the surface.

The increase in clay content from about 35% at depth to 55% close to the surface probably reflects surface leaching of carbonate as well as test variables, such as flocculation, that are difficult to control with soils rich in carbonate.

3.4/ Clay Mineralogy

The clay mineralogy of the oxidized crustal soils has been thoroughly discussed by Ogunbadejo and Quigley 1974 [11], Quigley and Ogunbadejo 1976 [13], and by Quigley 1975 [12], in OJT-CRP Project L-3 (Research Report 215). Briefly, oxidation of chlorites in the original soils reduces the charge deficiency to a level such that the clays swell forming an active smectite or montmorillonite. The process is one of Fe^{2+} oxidation to Fe^{3+} as discussed by Fanning and Jackson 1966 [4]. A discussion of clay mineralogy, including schematic illustrations of the constituent clays of the Sarnia soils, is given in Appendix D.

The amount of smectite increases from about 3% in the original, unweathered till to amounts as high as 15% near the surface depending on the intensity of oxidation and mode of deposition.

Typical X-ray diffraction traces are given in Figures 7 and 8 for the < 2 μm soil fraction. Figure 7 comes directly from Quigley, 1975 (Project L-3) [12] and shows clearly the abundant swelling clay in the sample from 0.3 m compared to a sample from 1.5 m depth. Figure 8 shows X-ray traces obtained for the two bulk samples of this investigation. The two curves labelled #1 for the natural wet soil indicate abundant smectite in the 0.3 m sample compared to the 2.4 m sample. It is significant that K^+ saturation efficiently collapses the smectite to 10 Å illite as shown by X-ray curves labelled #4. This indicates that the smectite is a relatively high charge variety, possibly with the charge deficiency located in the tetrahedral sheets so that K^+ is easily fixed.

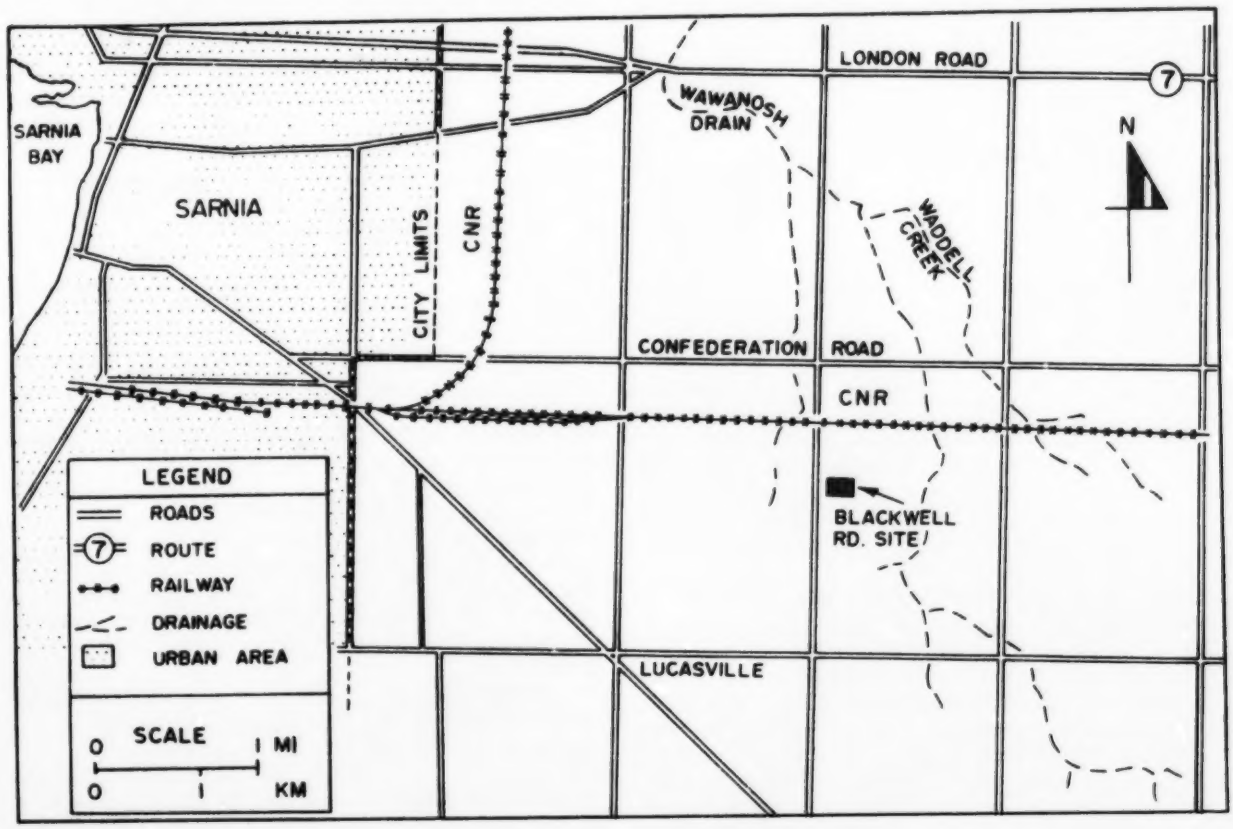

Figure 1/ Site Locations

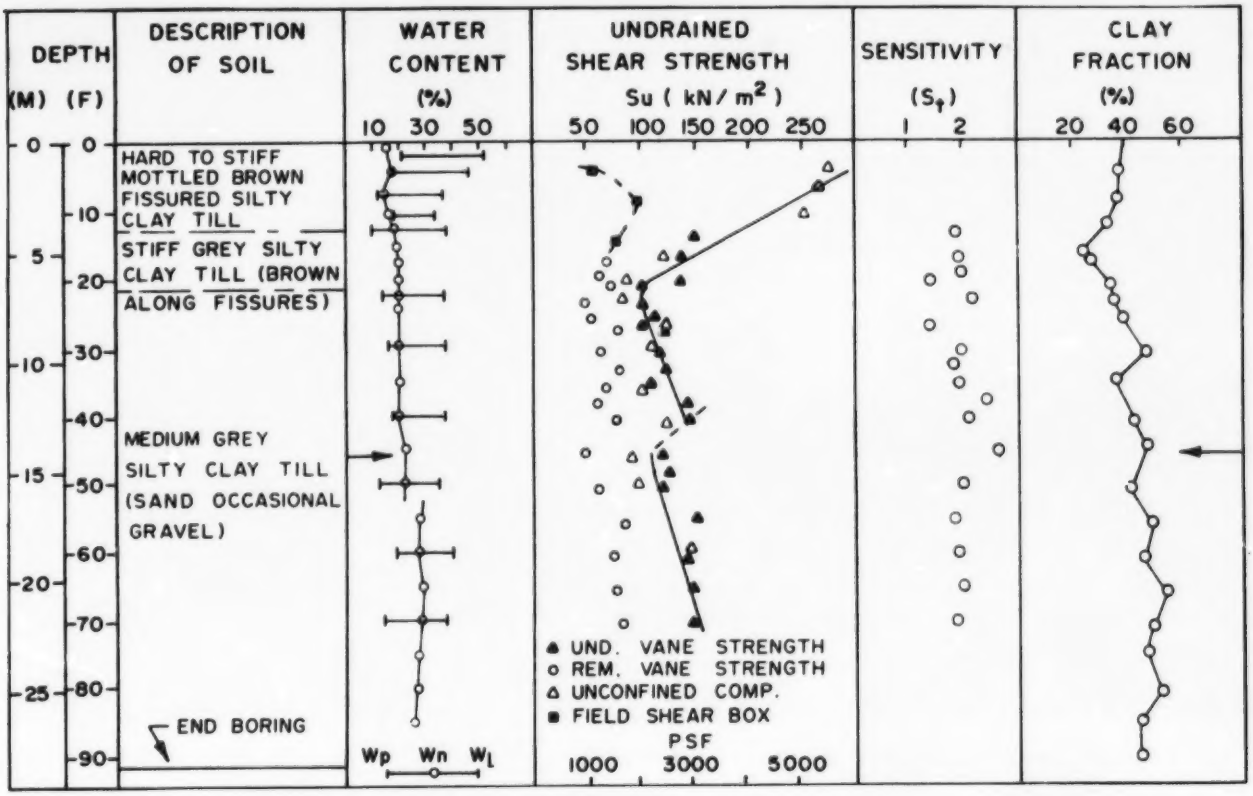

Figure 2/ Summary Plot, Engineering Properties vs Depth, Sarnia Clay Tills (Arrows mark probable contact between tills) Directly from Quigley and Ogunbadejo, 1976

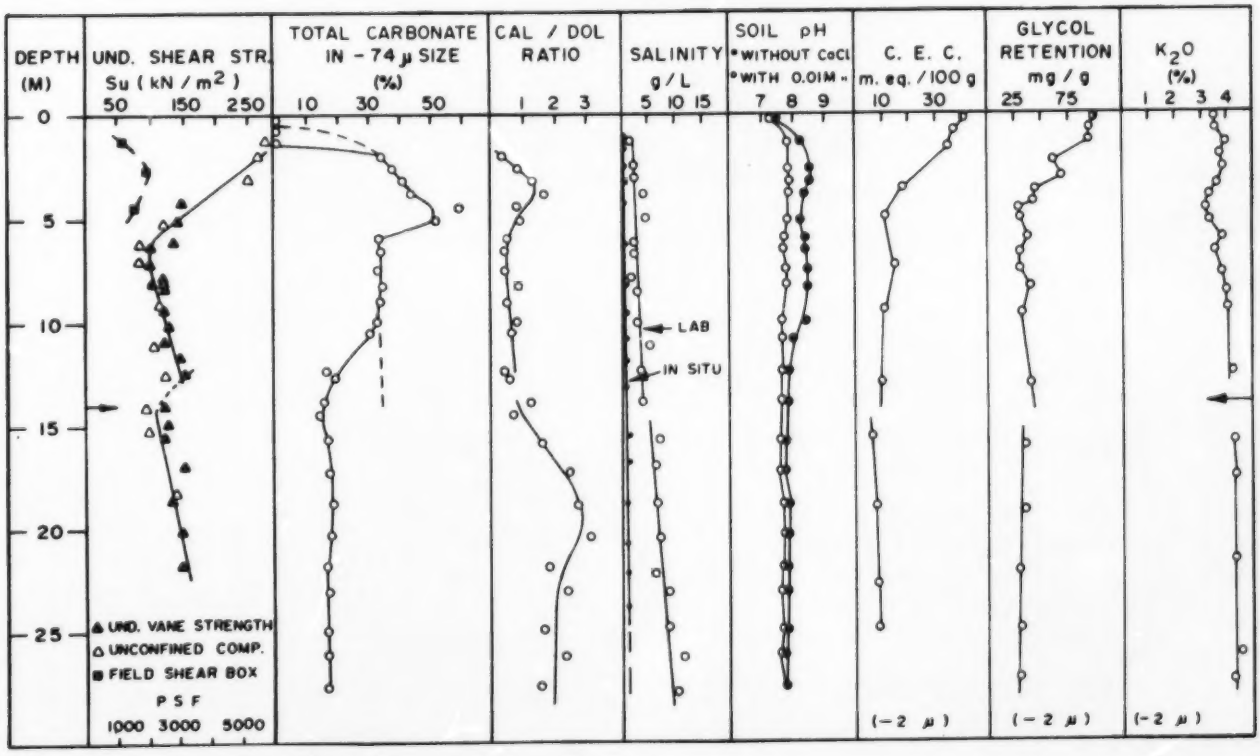

Figure 3/ Summary Plot, Physico-Chemical Properties vs Depth, Sarnia Clay Tills (Arrows mark probable contact between tills) Directly from Quigley and Ogunbadejo, 1976

Figure 4a/ Test Trench Dug by Backhoe at Sarnia City Landfill Site

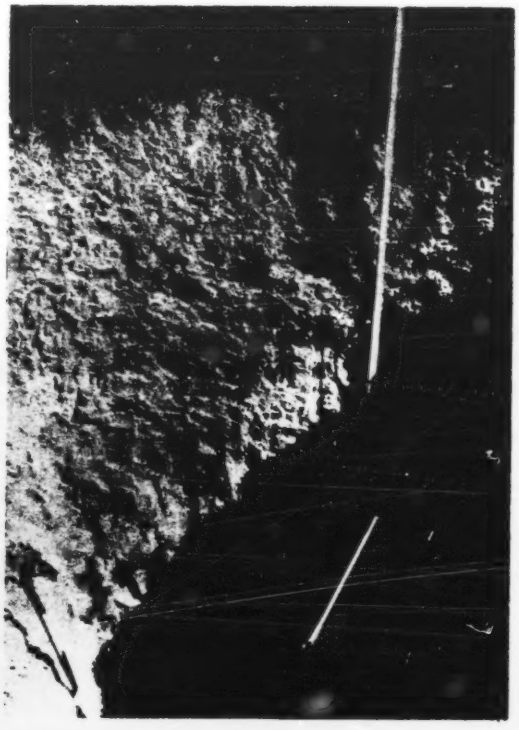

Figure 4b/ Close-up View of Test Trench

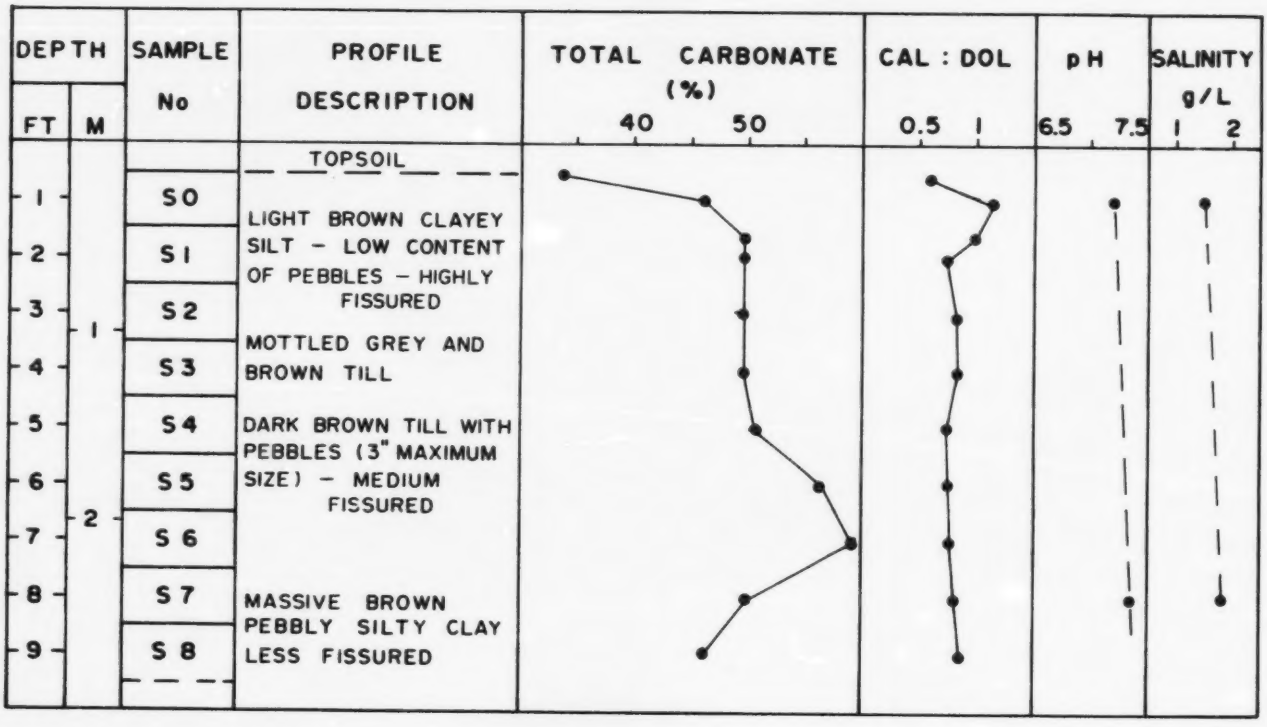

Figure 5/ Physico-Chemistry vs Depth, Sarnia Soils

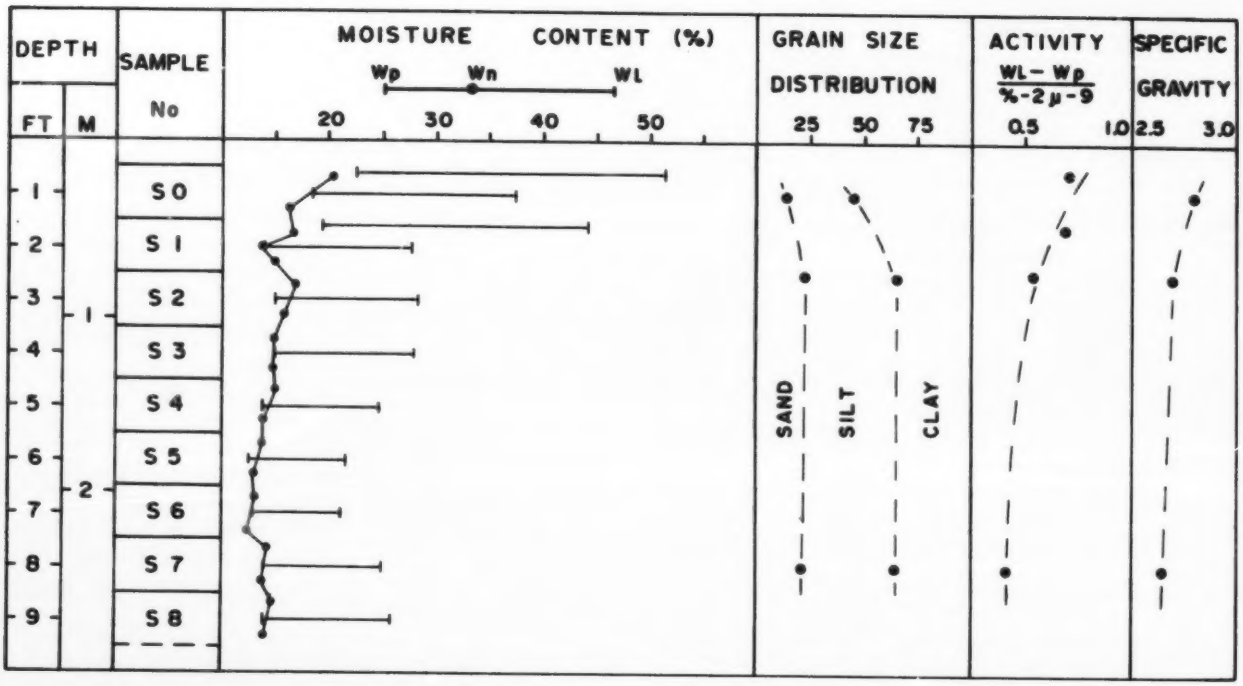

Figure 6/ Index Tests vs Depth, Sarnia Soils

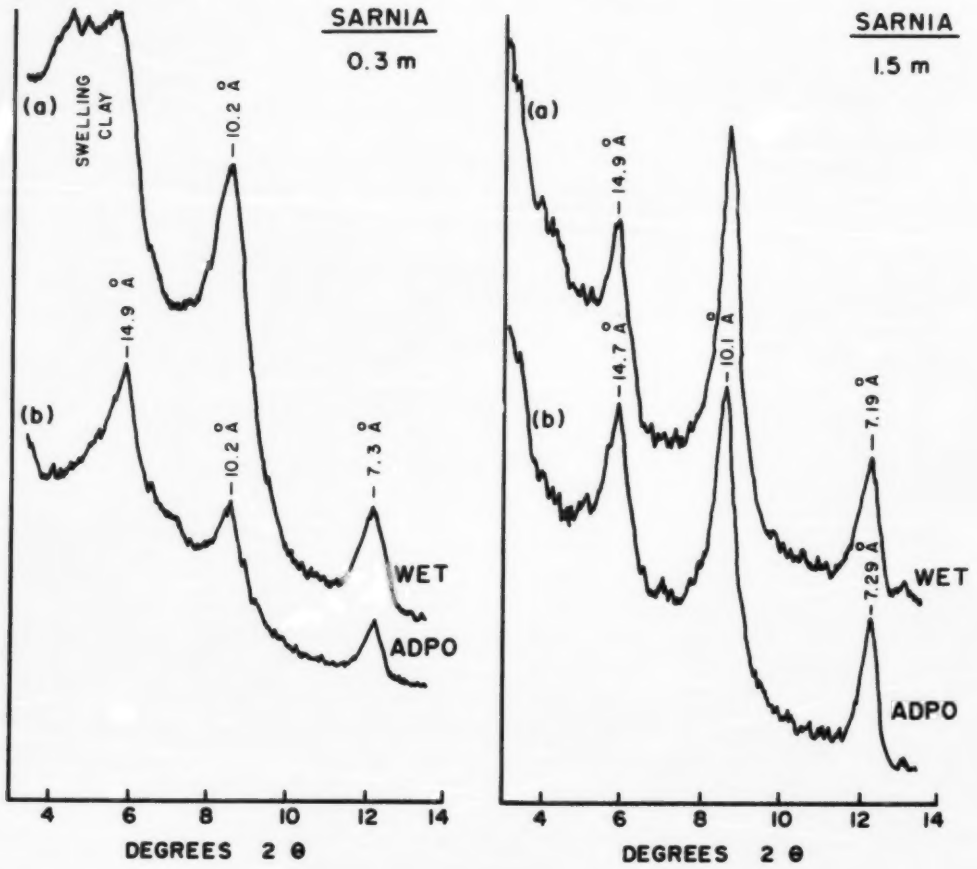

Figure 7/ X-Ray Diffraction Traces of Sarnia, Minus 2 Micron Clay Samples from 0.3 m and 1.5 m Depths; (a) Wet, (b) Air Dried; Preferred Orientation

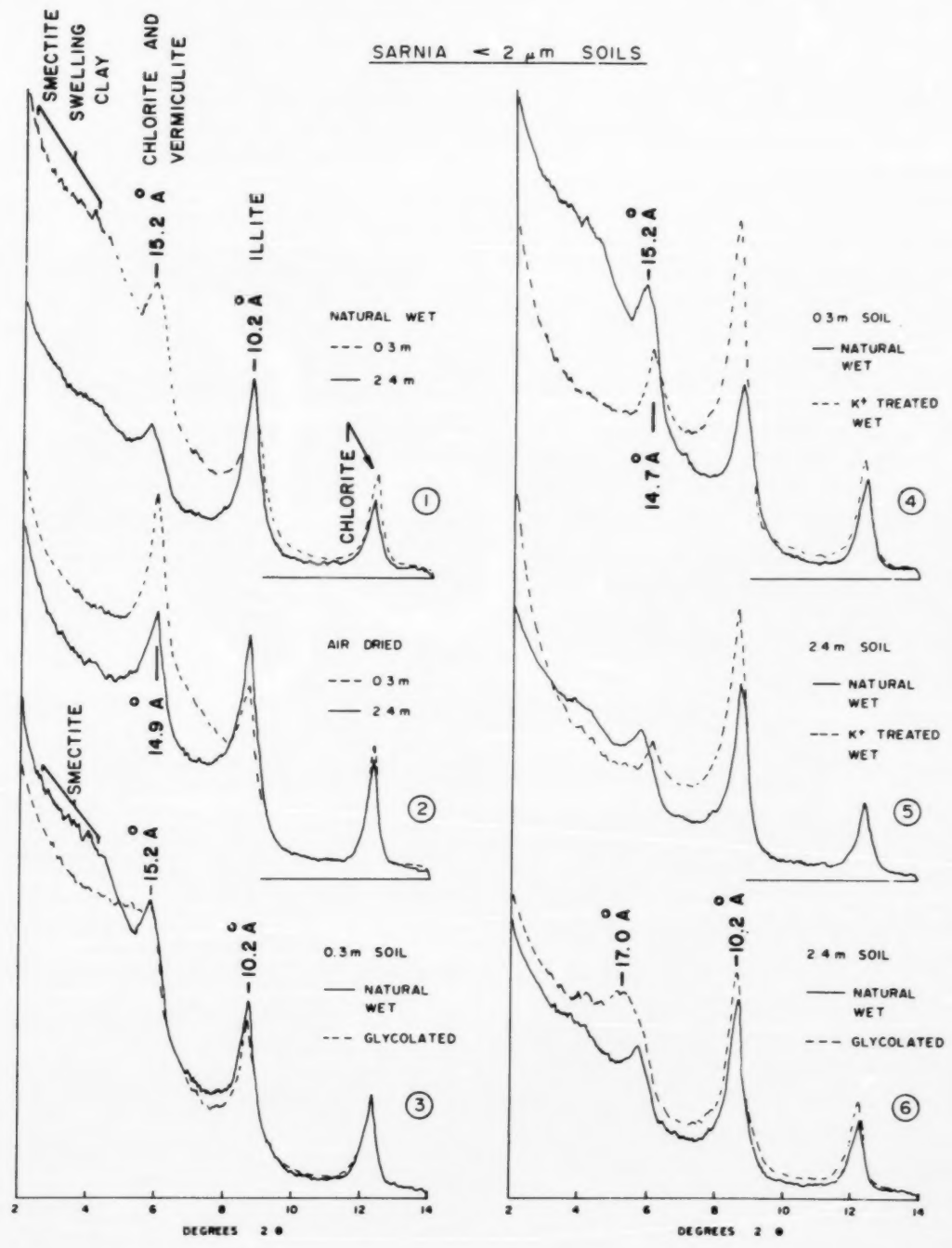

Figure 8/ Comparison Between X-Ray Traces of 0.3 m and 2.4 m Soils, Sarnia. Minus 2 μm. All Preferred Orientation

4/ Review of Previous Compaction Project L-3

4.1/ Introduction

Compaction Project L-3 [12], submitted on January 31, 1975 dealt extensively with the compaction-strength characteristics of the weathered surface clays in southwestern Ontario. The weathered, brown, desiccated, crustal soils were studied at three sites; Glencoe, Warwick and Sarnia. All sites showed the same weathering and mineralogical trends with depth through the crust as already discussed in Chapter 3, Site Geology and Mineralogy.

In this part of the report, the compaction-strength-swelling characteristics reported in Project L-3 are briefly reviewed prior to presentation of the cement and lime stabilization results of this Project L-6.

4.2/ Compaction-Strength Review

The compaction density-moisture relationships established for the upper crust at Sarnia are given in Figure 9. The figure clearly shows that the near surface soils high in smectite compact much less efficiently to about 1700 kg/m^3 (106 p.c.f.) at 0.3 m depth as compared to 1920 kg/m^3 (120 p.c.f.) at 1.8 m depth.

The values for optimum moisture content correspondingly decrease with depth from 21% near the surface to 15% at depth. Reference to the moisture content profile in Figure 6 indicates that the natural *in situ* moisture contents increase from about 13% at 2 m to 18% at 0.3 m depth. The *in situ* values, therefore, seem to be about 2% less than the optimum values obtained by Harvard Miniature.

In spite of the large variations in optimum density, the unconfined compression strengths at maximum density and optimum moisture are almost the same, as shown in the tabulation below:

Depth	ρ_d(max)	ω_{opt}	Unconf. Comp. Str.	ϵf
0.3 m	~1720 kg/m^3	~20%	~360 kPa	~ 9%
1.8 m	~2000 kg/m^3	~15%	~340 kPa	~14%

The failure strains (at peak strength) are significantly different, the near surface smectite-rich soils being more brittle in spite of their lower density and higher moisture content. These factors are believed to reflect the greater moisture deficiency and capillary stresses that probably exist in the smectite-rich near surface soils.

4.3/ Swell Potential and Swell Tests

Swell potentials, calculated using the method of Seed *et al*, 1962, are compared with the measured swell potentials and activity values in Figure 10 which is taken directly from Project L-3.

The measured swell, on compacted soil from 0.3 m depth, amounted to about 6% under a confining stress of 6.9 kPa (1 p.s.i.). This is low compared to the calculated values of over 12% predicted from the equation:

$$S = (3.6 \times 10^{-5})\,(A_{c2.44})\,(C_{3.44}).$$

This probably reflects the presence of abundant soil carbonate which acts as a strong inhibitor to the swelling of smectite-rich soils.

It would be quite useful to develop a swelling potential equation for the soils of southwestern Ontario that reflects the influence of the *in situ* clay mineralogy and soil carbonate, however, this would require another research project.

Swelling tests run on the $< 2\ \mu m$ fraction of the Glencoe soils free of carbonate still did not swell beyond about 8% until the absorbed Ca^{++} cations were exchanged with Na^{+} yielding heaves in excess of 24%.

Even with maximum measured swell potentials of only 6% at Sarnia, it was decided that this was significant and that the effects of cement and lime stabilization on swell should be established by Project L-6.

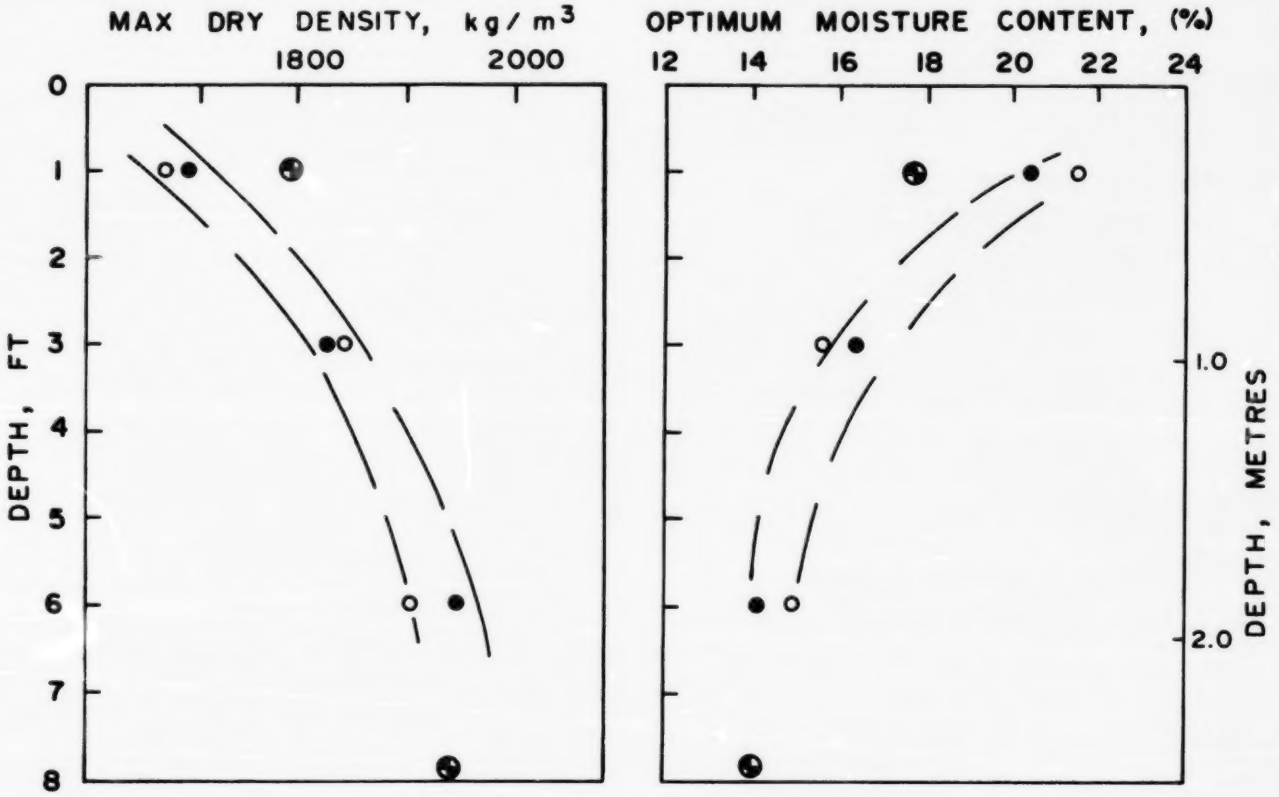

Figure 9/ Compaction Density-Moisture Relationships, Sarnia Clay Crust

● Air Dried; ○ Never Dried; Harvard Miniature (1975 Report), ◉ This Study, 1976

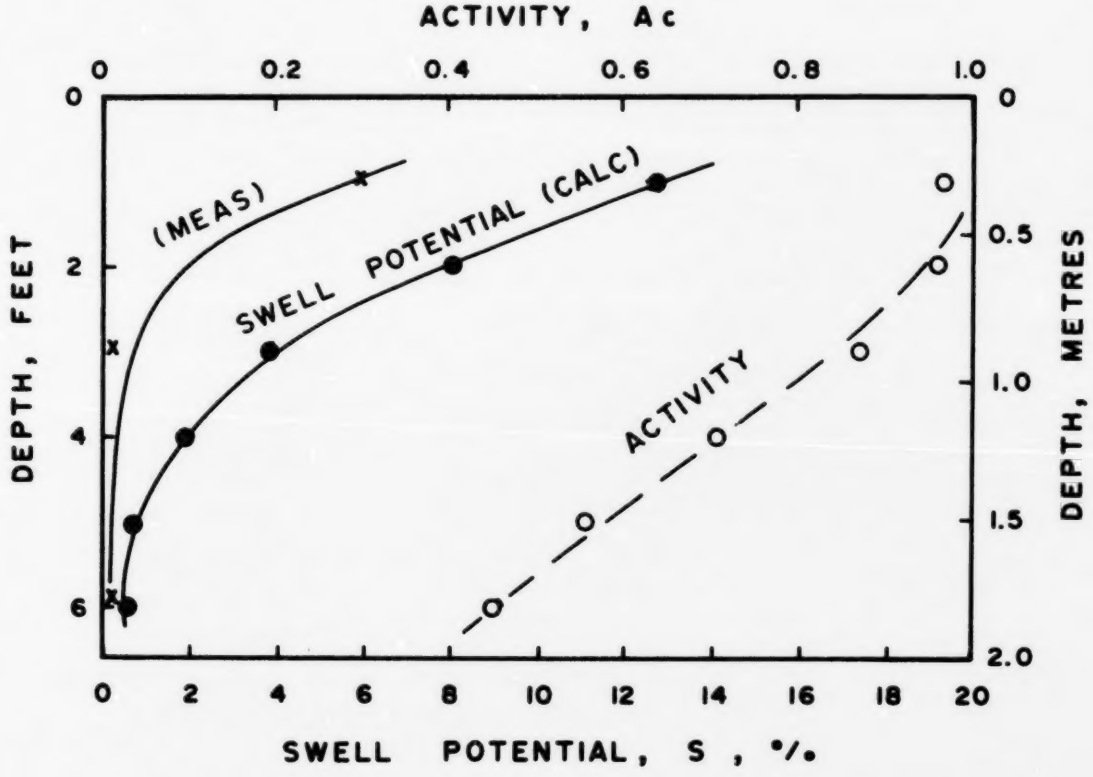

Figure 10/ Activity and Swells Potential Sarnia Soils (Calculated and Measured)

5/ Compaction Test Results and Discussion

5.1/ Composition of Test Materials

The composition of the two bulk samples and the Portland cement used for this study are summarized below:

Test Samples - Mineralogical and chemical analyses carried out on the two bulk samples yielded the following soil compositions and other chemical characteristics.

	0.3 m depth	2.4 m depth
Liquid limit, ω_ℓ %	52	25
Plastic limit, ω_p%	23	14
Activity, Ac	0.7	0.4
Specific Gravity, G_s	2.76	2.65
Sand Fraction, %	13	22
Silt Fraction, %	32	43
Clay Fraction, %	55	35
Carbonates, %	~ 40	~ 51
Illite, %	35 - 45	20 - 25
Chlorite, %	35 - 45	15 - 20
Smectite, %	~ 8	~ 1
Quartz and Feldspar, %	15 - 20	10 - 15
*Glycol Retention, mg/g	70	51
*C.E.C., m.eq./100 g	43	30
**K_2O, %	4.1	4.2

* Obtained on the < 2 μm fraction with carbonates.
**Obtained on the carbonate-free < 2 μm fraction.

Portland Cement - A bag of Type I Portland cement was obtained courtesy of St. Mary's Cement Company and had the following chemical composition as supplied by Mr. John Queenan, plant chemist:

Loss of Ignition		1.20%
$Si0_2$		20.08%
Fe_20_3		3.01%
$A1_20_3$		4.85%
Total CaO		62.97%
Free CaO		0.92%
MgO		3.82%
SO_3		2.96%
K_2O		0.96%
Combined	C_3S	54.6%
	C_2S	16.5%
	C_3A	7.8%
	C_4AF	9.2%

Hydrated Lime - The hydrated lime was a commercial grade variety obtained from a building supply retail outlet. Although the lime was not analyzed for its chemical composition, it is believed to contain a minimum of 55% of CaO + MgO in a partially hydrated state. Normal impurities in limes are non-dehydrated carbonates, and silica and alumina, probably in the form of quartz, feldspar and clay minerals. These impurities are not considered in any way deleterious to the usage of lime for stabilization [10].

5.2/ Compaction of Natural Soils

Compaction curves for the two bulk samples are given in Figure 11. Maximum density for the 2.4 m sample is about 1950 kg/m^3 (122 p.c.f.) compared to about 1800 kg/m^3 (112 p.c.f.) for the 0.3 m sample. Earlier tests on the profile yielded densities as low as 1750 kg/m^3 (109 p.c.f.) on the 0.3 m sample. It is quite clear that there is a significant difference in efficiency of compaction, the smectite-rich surface soils compacting to lower densities at correspondingly higher optimum water contents.

These densities are plotted in Figure 9 and clearly fall on the same trends determined in Project L-3. On this basis, the bulk samples taken from the trench were assessed as suitable for cement and lime stabilization experimentation.

5.3/ Compaction of Lime-Soil Mixtures

The compaction results for samples of whole soil-lime mixtures are given in Figures 12 and 13, and in Table 1. It is apparent from Figure 12 that addition of lime shifts the compaction curves down to lower densities and causes an increase in the required moisture contents for compaction.

Table 1/ Compaction Data for Whole Soil and Soil-Stabilizer Mixtures

Type of Stabilizer	Amount of Stabilizer %	0.3 m Depth γ_d max (pcf)	0.3 m Depth ρ_d max (kg/m^3)	0.3 m Depth ω opt (%)	2.4 m Depth γ_d max (pcf)	2.4 m Depth ρ_d max (kg/m^3)	2.4 m Depth ωopt (%)
Lime	0	111.7	1790	17.0	121.5	1945	14.0
	2	110.5	1770	19.0	117.0	1874	15.5
	4	104.0	1666	19.0	111.5	1786	17.5
	8	108.0	1730	19.5	113.5	1818	17.2
	12	101.0	1618	21.5	107.5	1722	18.5
Cement	0	111.7	1790	17.0	121.5	1945	14.0
	2	113.5	1818	18.5	118.3	1895	14.5
	4	116.5	1866	16.0	119.8	1919	14.2
	8	115.0	1842	18.0	120.2	1925	14.5
	12	117.7	1885	16.0	121.0	1938	14.5

The data for the 2.4 m soil are shown in Figure 13, and again it is apparent that addition of lime shifts all the compaction curves down to lower ρ_d values and increases the optimum moisture content.

The reversal of position noted for the 4 and 8% lime curves probably represents mixing variables caused by different operators for these two tests.

The summary plot of ρ_d max vs. percent hydrated lime in Figure 14 clearly shows the decline in ρ_d max obtained with increasing lime. For the 2.4 m soil, ρ_d max decreases from about 1945 kg/m^3 (121.5 p.c.f.) at 0% lime to about 1722 kg/m^3 (107.5 p.c.f.) at 12% lime. For the smectite-rich soil from 0.3 m depth, ρ_d max decreases from 1790 kg/m^3 (112 p.c.f.) at 0% lime to 1618 kg/m^3 (101 p.c.f.) at 12% lime.

5.4/ Compaction of Cement-Soil Mixtures

Compaction curves for the cement-soil mixtures are given in Figures 15 to 17, inclusive. In Figure 15 for the 0.3 m soil it is apparent that addition of cement tends to shift the compaction curves upwards to higher densities at slightly reduced values of ω_{opt}.

In Figure 16 for the 2.4 m soil, all compaction curves are essentially identical in shape and position in spite of variations in cement content from 0 to 12%.

The summary plot in Figure 17 shows a nearly constant maximum density for the 2.4 m soil at about 1925 kg/m^3 (120 p.c.f.) independent of cement content. The curve for the 0.3 m soil shows an abrupt increase in ρ_d max from 1790 kg/m^3 (112 p.c.f.) at 0% cement to 1866 kg/m^3 (116.5 p.c.f.) at 4% cement with constant ρ_d max at higher cement values.

5.5/ Compaction of < 2 μm Fractions

All compaction curves carried out on the < 2 μm soil fractions are plotted in Figure 18. The results for the non-stabilized soils are surprising in that the ρ_d max values are nearly the same for the 0.3 and 2.4 m depths (1474 kg/m^3 and 1502 kg/m^3, respectively). This is in spite of the very high smectite content of the 0.3 m sample.

Addition of both lime and cement at the 4% level causes a reduction in density and shifting of the curves to higher ω_{opt} values. Maximum densities for the lime stabilized samples are identical at ρ_d max = 1329 kg/m^3 and ω_{opt} = 37 to 38%. The addition of 4% cement causes a lesser reduction in maximum density to values of 1434 kg/m^3 and 1377 kg/m^3 for the 0.3 and 2.4 m depths, respectively.

Table 2/ Compaction Data
< 2 μm Fractions

Amount of Stabilizer (%)	0.3 m Depth			2.4 m Depth		
	γ_d max (pcf)	ρ_d max (kg/m^3)	ω opt (%)	γ_d max (pcf)	ρ_d max (kg/m^3)	ω_{opt} (%)
0	92.0	1474	29.2	93.8	1502	30.2
4% Cement	89.5	1434	36.8	86.0	1377	34.2
4% Lime	83.0	1329	37.2	83.0	1329	38.2

5.6/ Discussion

It is clear from Figure 9 that the bulk samples used for the stabilization study are similar to those previously studied from 0.3 and 2.4 m depths. The difference in maximum density is 150 kg/m^3 (~10 p.c.f.), a feature explained primarily by the high smectite (swelling clay) content of the near surface soils.

Addition of lime appears to cause the equivalent of soil flocculation due to increased pore fluid salt concentration and possibly due to Ca^{++} adsorption. This results in resistance to compaction, and thus lower values of maximum density at higher optimum water contents, as reported by many authors including Ladd *et al*, 1960 [9]. In this study, both the shallow and deeper crustal soils behaved similarly as shown in Figure 14.

Addition of cement appears to cause a dispersion of the soil, generally resulting in improved compaction. This feature was discussed in some detail by Croft, 1967 [1], who observed that the maximum compacted density increased with increasing cement (up to 20%) for soils composed of halloysite, kaolinite, illite and illite-montmorillonite. Only in the case of nearly pure montmorillonite did the maximum density decrease. In this study, the maximum density of both soils remained either constant (2.4 m) or increased (0.3 m) with increasing cement content so the results are in accordance with previous work.

The results for the < 2 μm soil fractions are difficult to interpret and not as expected. Although the maximum density values are much lower than those for the whole soil (~ 1450 kg/m^3 compared to 1800-1950 kg/m^3), there is no significant spread between the 0.3 m and 2.4 m samples. Also, addition of cement caused a decrease in maximum density, quite unexpected from the whole soil results and the literature review.

It is possible that these anomalies are related to:

a/ the fractionation procedures which probably removed considerable fine carbonate,
b/ the variable amount of Ca^{++} used for flocculation, and most importantly,
c/ the effects of air drying these clays before compaction.

As discussed by Ogunbadejo and Quigley, 1974 [11], air-drying may cause significant permanent collapse of the smectite phase plus agglomeration of the particles which may increase the maximum density well above values for never-dried soil. This would close the "gap" between the 0.3 and 2.4 m soils in Figure 18.

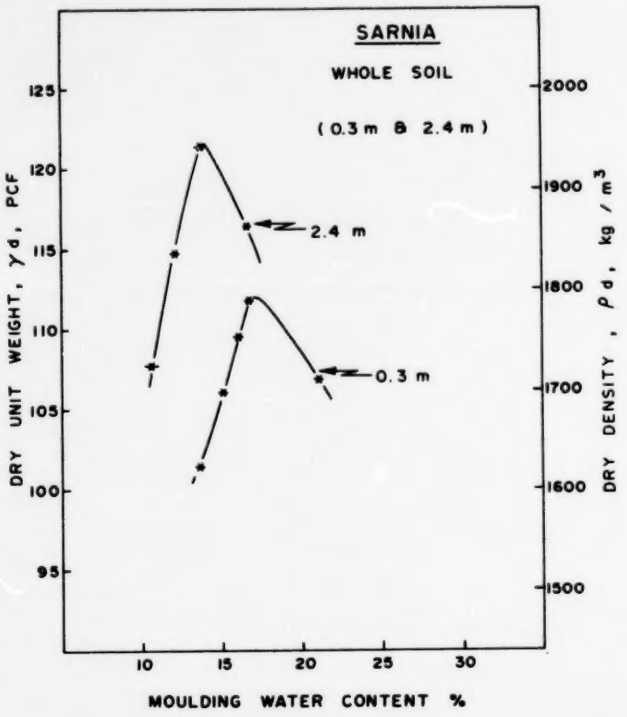

Figure 11/ Harvard Miniature Compaction Curves for Sarnia Whole Soil, 0.3 m & 2.4 m Depth. 0% Stabilizer

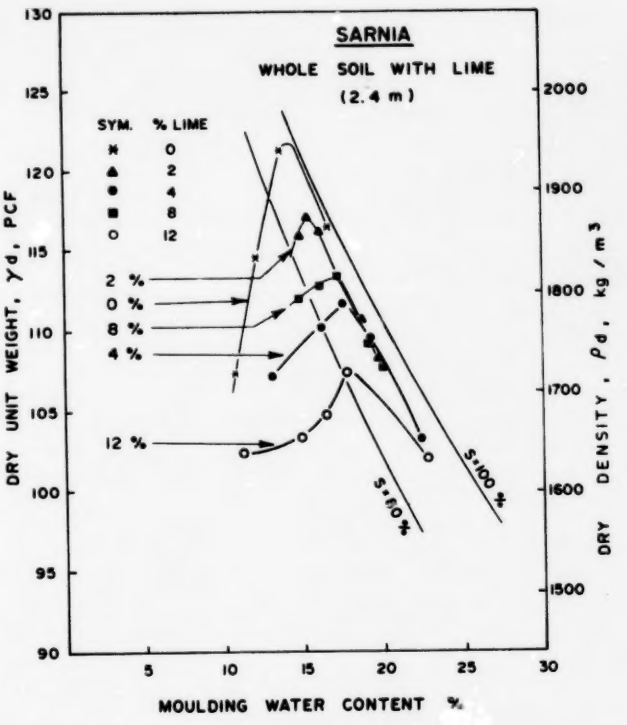

Figure 13/ Harvard Miniature Compaction Curves for Sarnia Soil, 2.4 m Depth, 0, 2, 4, 8, and 12% Lime

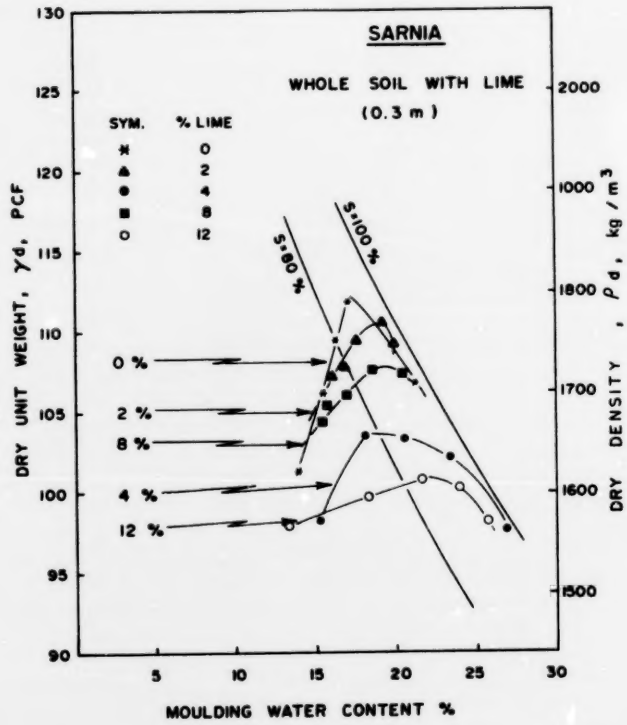

Figure 12/ Harvard Miniature Compaction Curves for Sarnia Soil, 0.3 m Depth, 0, 2, 4, 8, and 12 % Lime

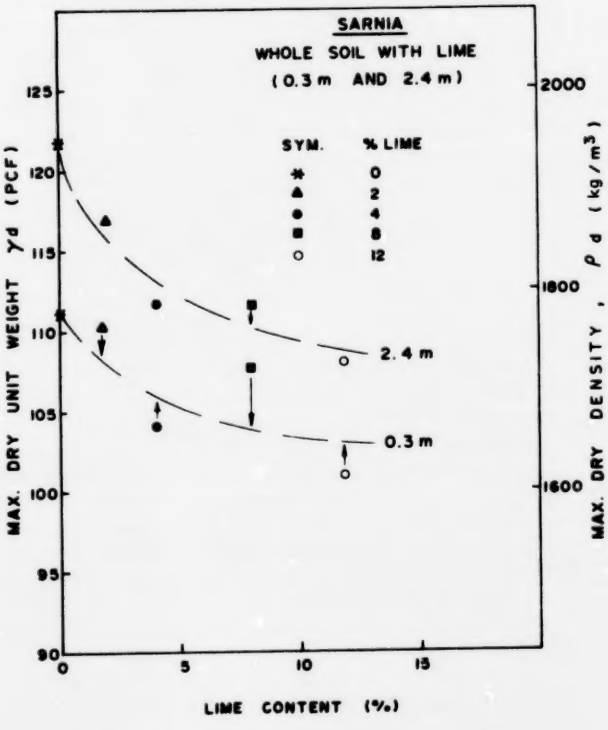

Figure 14/ Maximum Dry Unit Weight vs Percentage Hydrated Lime

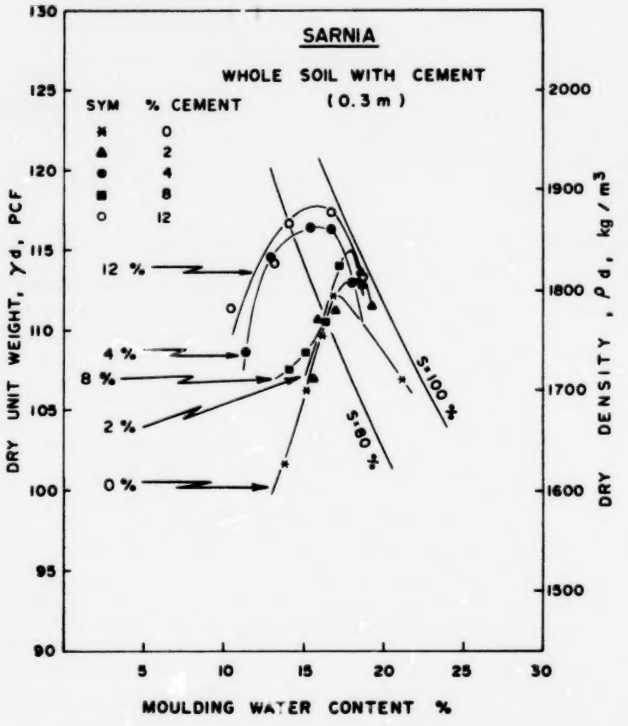

Figure 15/ Harvard Miniature Compaction Curves for Sarnia Soil, 0.3 m Depth, 0, 2, 4, 8, and 12 % Cement

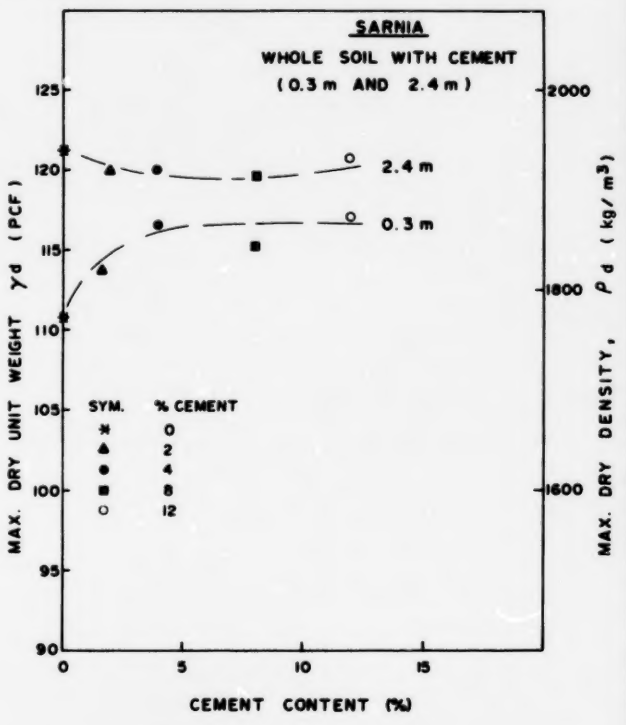

Figure 17/ Maximum Dry Unit Weight vs Percentage of Cement

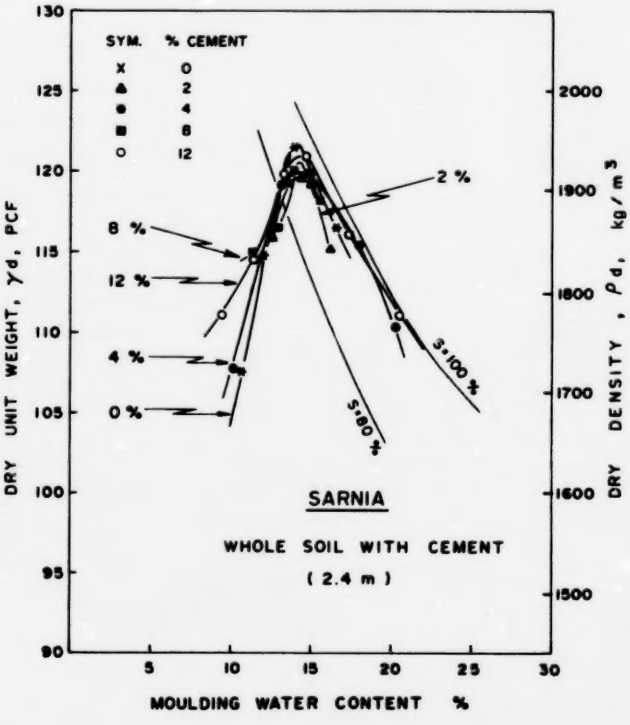

Figure 16/ Harvard Miniature Compaction Curves for Sarnia Soil, 2.4 m Depth, 0, 2, 4, 8, and 12 % Cement

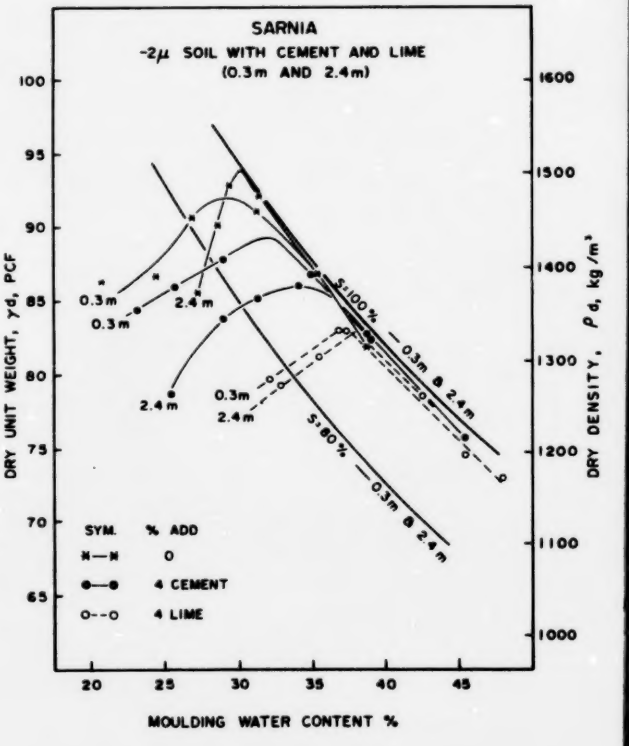

Figure 18/ Harvard Miniature Compaction Curves for Sarnia -2μ Soil 0.3 m and 2.4 m Depth, 0 and 4 % Lime and Cement

6/ Unconfined Compression Results and Discussion

The unconfined compression test results are presented in Tables 3 and 4, and in Figures 19 to 31 inclusive. Included in this section are the results for both stabilized whole soil and the < 2 μm fractions.

Table 3/ Compressive Strength of Whole Soil-Lime Mixtures

Depth (m)	Lime (%)	Maximum Compressive Strength											
		1 day		3 days		7 days		28 days		60 days		90 days	
		kPa	psi	kPa	psi	kPa	psi	kPa	psi	kPa	psi	kPa	psi
0.3	0	-	-	-	-	214	31	221	32	214	31	-	-
	2	-	-	-	-	593	86	655	95	696	101	738	107
	4	-	-	-	-	338	49	469	68	676	98	758	110
	8	-	-	-	-	800	116	1103	160	1365	198	1345	195
	12	-	-	-	-	621	90	1220	177	1503	218	1593	231
2.4	0	-	-	-	-	269	39	297	43	317	46	-	-
	2	-	-	-	-	517	75	924	134	1193	173	1145	166
	4	-	-	-	-	414	60	910	132	1386	201	1407	204
	8	-	-	-	-	531	77	938	136	1489	216	1634	237
	12	-	-	-	-	365	53	979	142	1379	200	1448	210

Table 4/ Compressive Strength of Whole Soil-Cement Mixtures

Depth (m)	Cement (%)	Maximum Compressive Strength											
		1 day		3 days		7 days		28 days		60 days		90 days	
		kPa	psi	kPa	psi	kPa	psi	kPa	psi	kPa	psi	kPa	psi
0.3	0	-	-	-	-	214	31	221	32	214	31	-	-
	2	-	-	-	-	552	80	758	110	862	125	896	130
	4	-	-	758	110	1379	200	1551	225	-	-	-	-
	8	-	-	-	-	1862	270	2655	385	2709	393	2979	432
	12	1593	231	-	-	3689	535	4268	619	-	-	-	-
2.4	0	-	-	-	-	269	39	297	43	317	46	-	-
	2	-	-	-	-	1117	162	1441	209	2020	293	1731	251
	4	1276	185	-	-	2158	313	3330	483	3323	482	-	-
	8	-	-	-	-	2482	360	4068	590	4826	700	4909	712
	12	2220	322	-	-	3710	538	5020	728	5309	770	-	-

6.1/ Strength of Natural Soils

The compressive strength of the compacted natural soil from 0.3 m depth was about 214 kPa (31 p.s.i) at a failure strain of 3 to 4%. The corresponding compressive strength of the compacted 2.4 m soil was 270 kPa (40 p.s.i.) at a failure strain of 2.5 to 3%. These two strengths are the reference strengths to which the stabilized soil strengths are referred.

6.2/ Strength of Lime-Soil Mixtures

Typical stress-strain curves for lime-soil mixtures are presented in Figures 19 and 20. Horizontal arrows indicate the failure strength of the equivalent, unstabilized soils. It is clear from the curves that the addition of lime markedly increases the strength and reduces the failure strain to about 1% from the original 3 to 4%. It is also apparent that the strength increases markedly with curing time in many test sequences.

A summary plot of the strength gained for each percentage of lime in mixtures of the 2.4 m soil at different curing times is shown in Figure 21. Lime caused a significant approximate fivefold increase in strength of this soil. It is particularly significant that 2 and 4% lime were nearly as effective as 8 and 12% lime in both magnitude and time rate of strength gain in these relatively smectite-free soils. A corresponding summary plot for the 0.3 m soil is given in Figure 22. Again, lime stabilization has markedly increased the strength, however, 2 and 4% lime have produced only a threefold increase in comparison to six to sevenfold increase for 8 and 12% lime. It is also significant for the stronger samples, that the strength increase is still continuing well beyond 60 days of curing time.

6.3/ Strength of Cement-Soil Mixtures

Typical stress-strain curves for the cement-soil mixtures are presented in Figures 23 and 24. The curves show a significant increase in strength and reduction in failure strain to about 1% from the original 3 or 4% for unstabilized soils.

The summary plot for the 2.4 m soil given in Figure 25 shows the efficiency of cement stabilization, a seventeenfold increase in strength developing in 60 days for 8 and 12% cement stabilizer added. Even with 2% cement, a sixfold increase in strength develops in about 60 days.

In Figure 26, the strength gains for the 0.3 m soil are plotted versus time. Although the strength gains are significant, they are considerably less than those for the deeper soil and reflect the presence of smectite in the 0.3 m soils. For example, 4% cement produces a compressive strength of about 3200 kPa for the 2.4 m soil compared to only about 1600 kPa for the 0.3 m, smectite-rich soil.

6.4/ Strength of < 2 μm Soil Fractions

Stress-strain curves for the < 2 μm soil fractions compacted with or without stabilizers are given in Figures 27 and 28. The ultimate strength of the unstabilized clays from 0.3 m and 2.4 m depth are 275 kPa and 496 kPa, respectively. Both samples failed by bulging although peak strength was reached at fairly low strains of 1 to 3%.

Addition of 4% lime to the fine fraction produced marked strength increases with time, as shown in Figure 27. A corresponding decrease in failure strain is also apparent. Peak strengths at 60 days for both depths are of similar magnitudes of about 1200 kPa.

Addition of 4% cement to the < 2 μm fractions produced similar trends, as shown in Figure 28. Interestingly, the 60-day strengths for both depths are about the same at about 1400 kPa, a value only slightly higher than those for the lime stabilized fines shown in Figure 27.

6.5/ Discussion

For purpose of discussion and comparison, summary plots of the stabilization data are presented in Figures 29 to 31. From the strength-time plot in Figure 29, it is apparent that:

1/ Cement is a much more effective stabilizer than lime in terms of strength gain at 60 days.

2/ The smectite-rich near surface soils (0.3 m depth) seem to be less efficiently stabilized than the deeper soils from 2.4 m depth.

3/ The rate of strength gain is much faster for cement than lime. The cement reactions seem essentially completed at 60 days, whereas the lime-soil reactions may still be proceeding at 60 days.

The plot of amount of stabilizer versus compressive strength in Figure 30 shows that continued addition of cement produces a continually greater 28 day strength for both soil depths. For lime stabilization, however, the strength increases above 4% added, seem to be minimal. In fact, even 2% lime seems nearly as effective as 8 and 12% lime for the testing procedures used in this research program.

The summary plot of the stabilization results on the < 2 μm soils is interesting indeed. Ultimate compressive strengths for both 4% lime and 4% cement are very similar at 60 days (approx. 1200 to 1400 kPa). Reference to Figure 30, shows that the clay fraction stabilized with lime is considerably stronger than the whole soil stabilized with lime. This is believed to indicate significant chemical interaction between the clay minerals and the hydrated lime to produce a fairly strong final product. Cementation, or stabilization by cement on the other hand, has been markedly inhibited in the clay-rich < 2 μm fractions yielding compressive strengths much lower than those obtained for whole soil samples as shown in Figure 29.

The results of this investigation conform reasonably well with the results of others. Croft, 1967 [1], Herzog and Mitchell, 1963 [6], and Eades *et al*, 1960 [3], all tested pure mineral mixtures, and their results confirm the difficulty in stabilizing smectites and the relatively greater efficiency of lime for stabilization of smectite-rich soils.

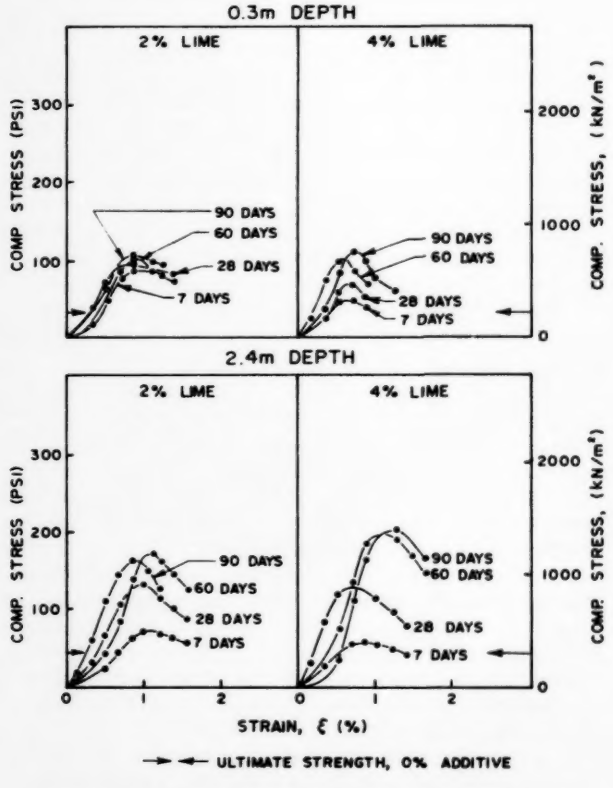

Figure 19/ Effect of Curing Time on Stress-Strain Behaviour of Sarnia Whole Soil-Lime Mixtures

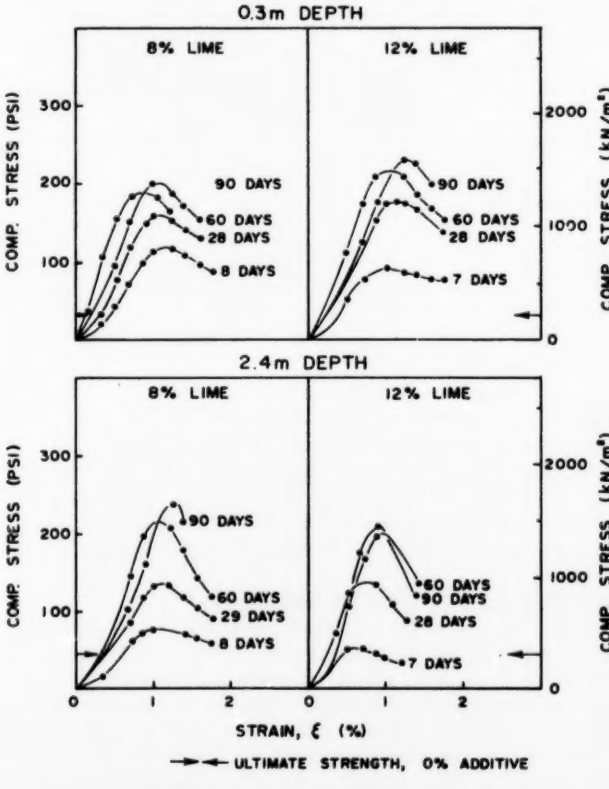

Figure 20/ Effect of Curing Time on Stress-Strain Behaviour of Sarnia Whole Soil-Lime Mixture

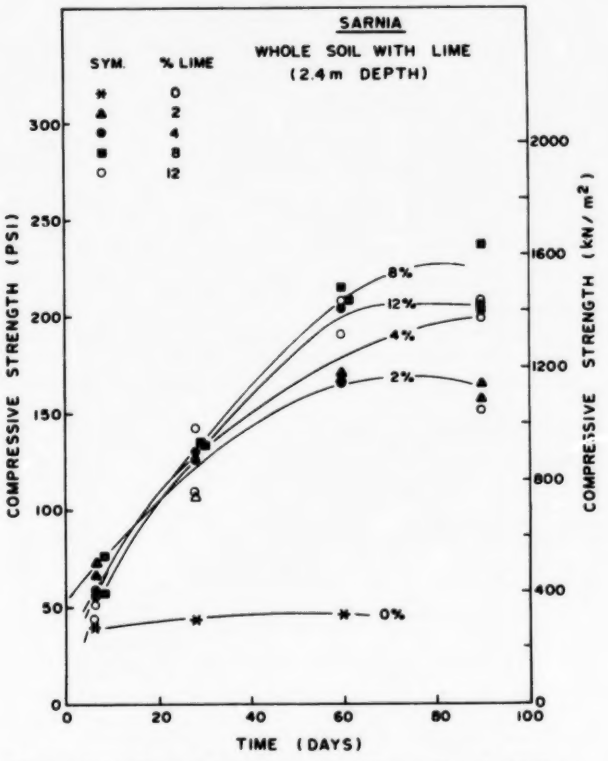

Figure 21/ Strength Gain with Time; Sarnia Soils, 2.4 m Depth; 0, 2, 4, 8, 12 % Hydrated Lime

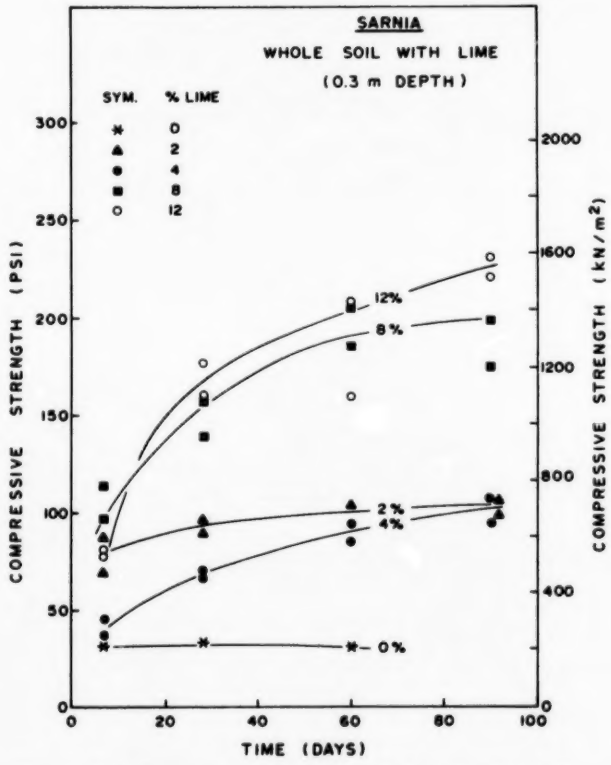

Figure 22/ Strength Gain with Time; Sarnia Soil 0.3 m Depth, 0, 2, 4, 8, 12 % Hydrated Lime

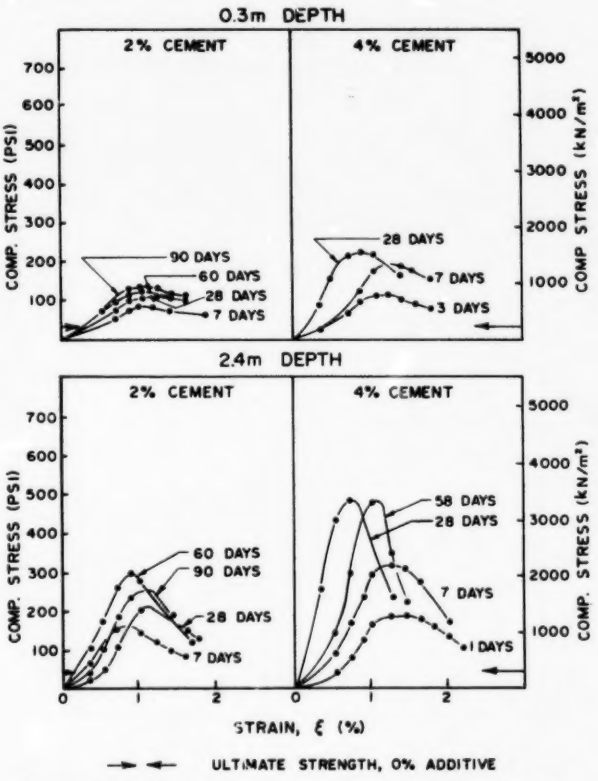

Figure 23/ Effect of Curing Time on Stress-Strain Behaviour of Sarnia Whole Soil Cement Mixture

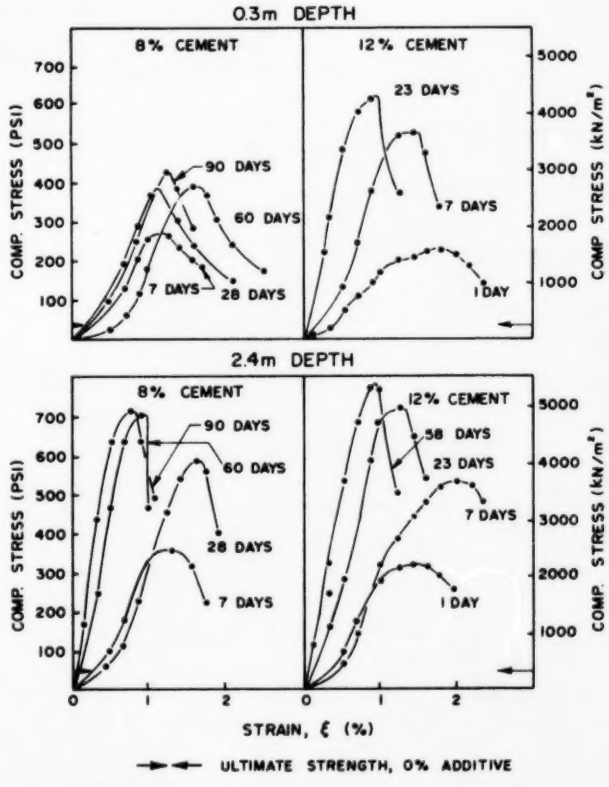

Figure 24/ Effect of Curing Time on Stress-Strain Behaviour of Sarnia Whole Soil Cement Mixture

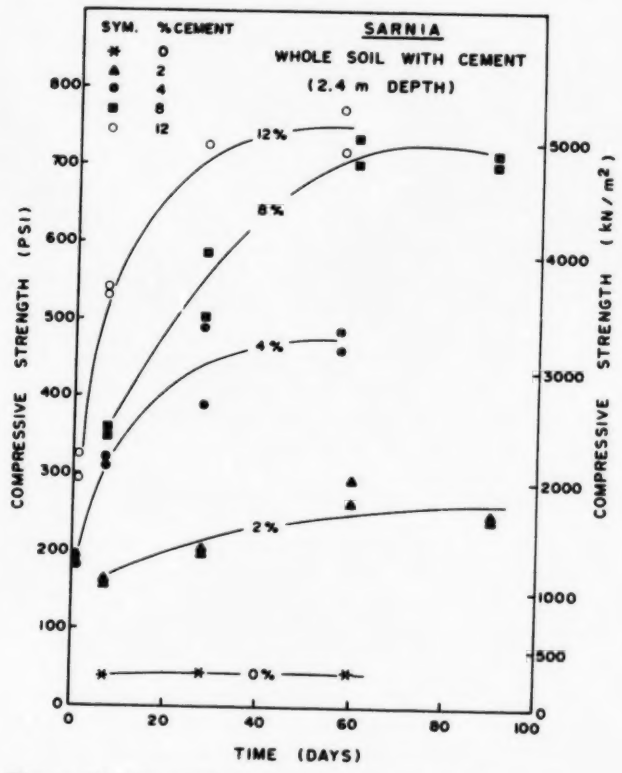

Figure 25/ Strength Gain with Time; Sarnia Soil 2.4 m Depth 0, 2, 4, 8, 12 % Cement

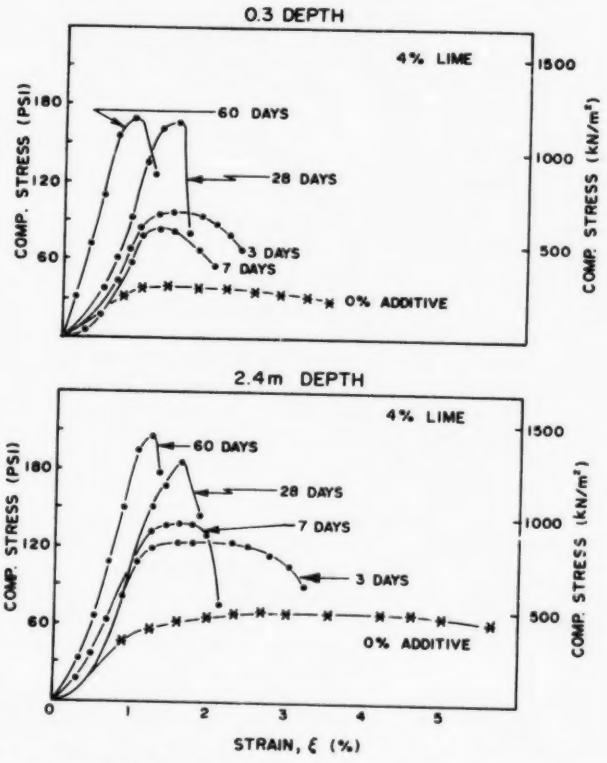

Figure 27/ Effect of Curing Time on Stress-Strain Behaviour of Sarnia <2μm Soil-Lime Mixture

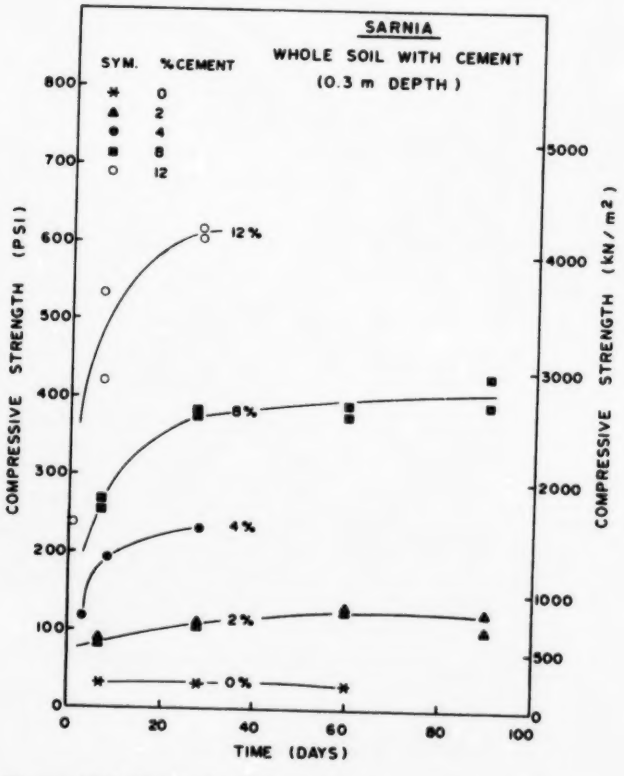

Figure 26/ Strength Gain with Time; Sarnia Soil, 0.3 m depth, 0, 2, 4, 8, 12 % Cement

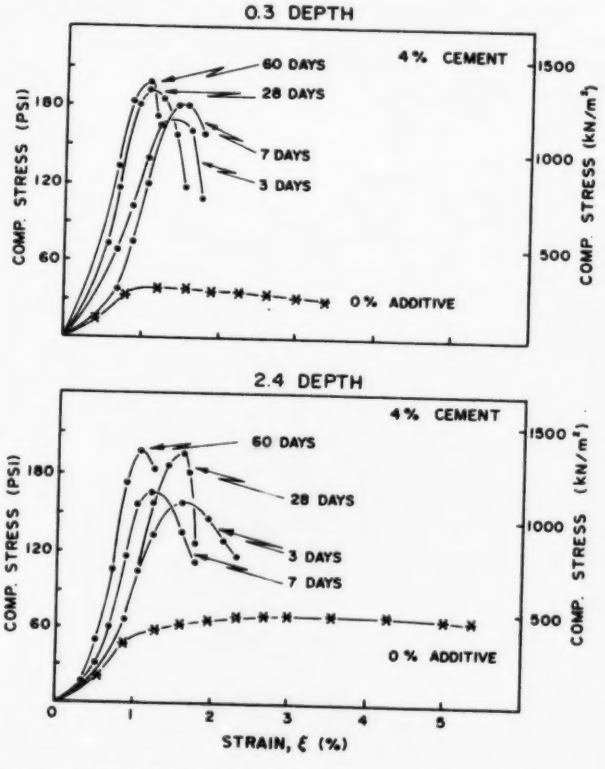

Figure 28/ Effect of Curing Time on Stress-Strain Behaviour of Sarnia <2μm Soil Cement Mixture

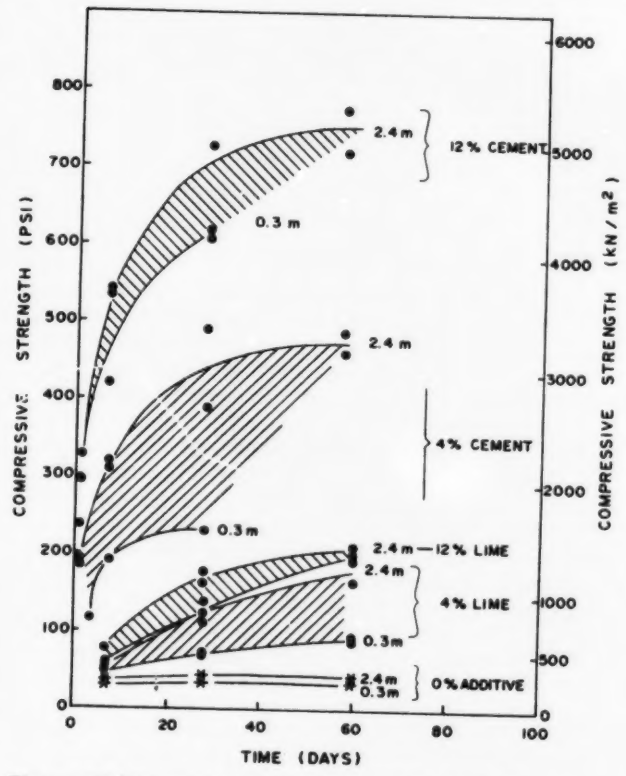

Figure 29/ Comparison of Lime and Cement Stabilization; 0, 4, 12 % Stabilizer Sarnia Soil from 0.3 m and 2.4 m Depth

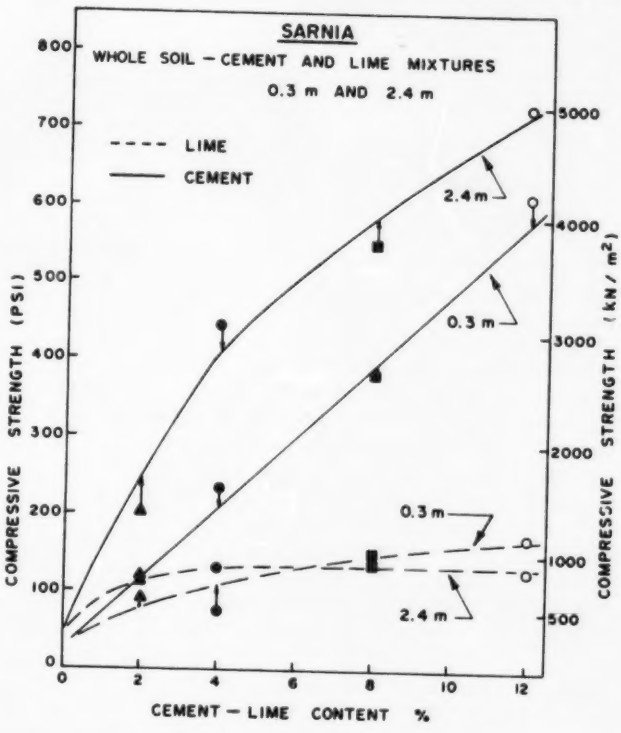

Figure 30/ Summary Plot of Compressive Strength, 28 days Curing Time vs Percentage of Cement and Lime, Sarnia Whole Soil 0.3 m and 2.4 m Depth

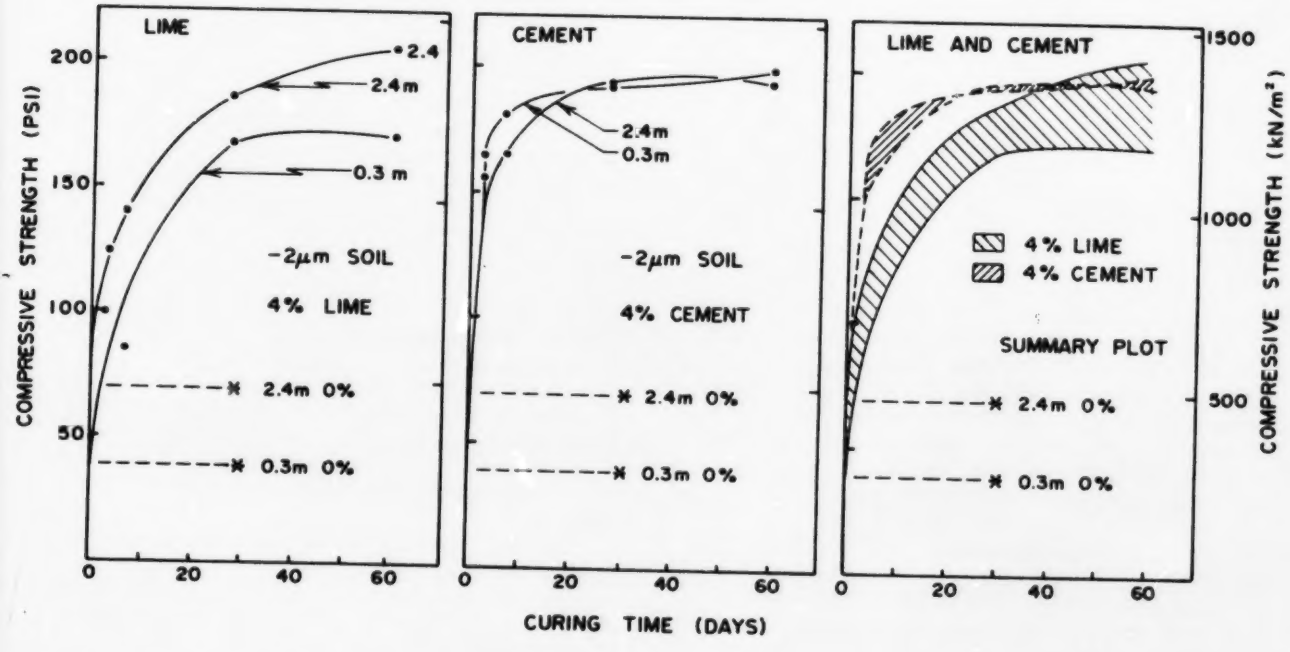

Figure 31/ Compression Strength vs Curing Time of <2μm Sarnia Soils With Lime and Cement Mixtures, 0% and 4% Additives, 0.3 m and 2.4 m Depth

7/ Swelling Test Results and Discussion

Swelling tests were carried out to determine the expansion characteristics of the soils before and after stabilization. Whole soils and < 2 μm soil fractions from 0.3 and 2.4 m depths were used with various admixtures of lime and cement. The swell tests were run on uncured and cured specimens to establish the effects of early soaking (i.e., field rainfall) prior to "setting" of the stabilizers. Swell tests on stabilized clay fractions were of special interest.

The swell test results as a function of elapsed time are presented in Figures 32 to 37 inclusive, and the values of ultimate swell are tabulated in Tables 5 and 6.

Table 5/ Ultimate Swell of Whole Soil With and Without Stabilizers

Stabilizer		0.3 m Soil Percent Swell (%)		2.4 m Soil Percent Swell (%)	
		Uncured	28-day cured	Uncured	28-day cured
Lime	0%	1.72	1.72	0.15	0.15
	4%	0.3	0.3	0.6	0.3
	12%	0.5	0.5	1.5	0.4
Cement	0%	1.72	1.72	0.15	0.15
	4%	1.4	0.2	0.5	0.2
	12%	4.5	0.3	1.1	0.4

Table 6/ Ultimate Swell of < 2 μm Soil Fractions

Stabilizer	0.3 m Soil Percent Swell (%)		2.4 m Soil Percent Swell (%)	
	Uncured	7-day cured	Uncured	7-day cured
0%	8.7	8.7	7.3	7.3
Lime 4%	1.0	0.2	2.8	0.2
Cement 4%	0.8	0.2	1.4	0.2

7.1/ Whole Soil Swelling

The swelling curves for the whole soil samples are plotted in Figures 32 through 35. Maximum or ultimate swell values for the unstabilized soils from 0.3 and 2.4 m amounted to 1.7 and 0.15%, respectively. It is obvious from Figure 32, however, that the bulk of the swelling had already occurred within 48 hours of the start of the tests.

7.2/ Whole Soil-Lime Swelling

The swelling vs. time curves for the uncured soil-lime mixtures are given in Figure 32, and it is clear that although there is some inhibition, the amount of swell is very significant if curing is not allowed. Note particularly the swell curve for the 2.4 m soil with 12% lime that reaches a swell value of 1.5% compared to 0.15% for the same soil free of stabilizer.

After 28 days of curing, the soil is effectively stabilized by lime as shown in Figure 34 for the 0.3 m soils. Maximum swell amounted to only 0.3 and 0.5% for 4 and 12% of lime additive compared to 1.7% without stabilizer.

7.3/ Whole Soil-Cement Swelling

Swell test results for the whole soil stabilized with cement are plotted in Figure 33 (uncured), and Figure 34 (cured 28 days). In comparison to the soils without added cement, it is obvious from Figure 33 that uncured cement acts like a dispersing agent causing swell far in excess of the natural values. Note particularly the sample from 0.3 m depth with 12% of uncured cement which swelled 4.5% compared with 1.72% without cement.

After curing, however, the soil appears once again to be effectively stabilized and final swells after about 900 h of soaking amounted to only 0.2 to 0.3% compared to 1.72% for unstabilized soil from 0.3 m depth. The deeper soil (Figure 35) swells 0.2 to 0.4% after prolonged soaking which, although small, is somewhat greater than the 0.15% obtained on its unstabilized equivalent.

7.4/ Swelling of < 2 μm Fraction

The swell test results obtained on compacted samples of the soil fines are presented in Figures 36 and 37 for uncured and 7-day cured specimens, respectively.

Extensive swells of 7 to 9% were recorded for the unstabilized fines with the 2.4 m soil swelling almost as much as the 0.3 m soil. Addition of 4% lime and 4% cement inhibited swelling even without curing, however, the ultimate swells of 0.8 to 2.8% are significant (Figure 36).

Seven days of curing effectively stabilized the clay fractions as shown in Figure 37, the ultimate swells amounting to only 0.2% for both lime and cement.

7.5/ Discussion

The compacted specimens of Sarnia soil containing admixtures of both lime and cement swell significantly if soaked prior to curing. Cement creates the most dramatic swelling, probably since it acts as a soil dispersant by adding hydroxyls to the clay minerals thus increasing their negative charge and enhancing double layer expansion and resultant swelling.

It is not known how uncured lime causes swelling over and above that for the original unstabilized soil, however, there is a possiblity that the volume increase is related to crystal growth of the hydrated lime itself.

Curing the samples at constant volume results in effective stabilization of the Sarnia soils and very little swell develops with time. The upturn in the

curves in Figures 34 and 35 are exaggerated by the logarithmic scale used.

The results for the fine fractions of the soils were somewhat surprising, in particular the extreme effectiveness of both 4% lime and 4% cement in preventing all swell after 7 days of curing. It appears again that the clay minerals in the Sarnia soils seem especially suitable for stabilization. One unknown factor is the reason for the relative inactivity of the 0.3 m specimen which contains abundant swelling clay especially in the < 2 μm fraction. As discussed previously, it is speculated again that air-drying the samples in order to collect the < 2 μm fraction may have collapsed the clay sufficiently and permanently enough to markedly reduce its activity.

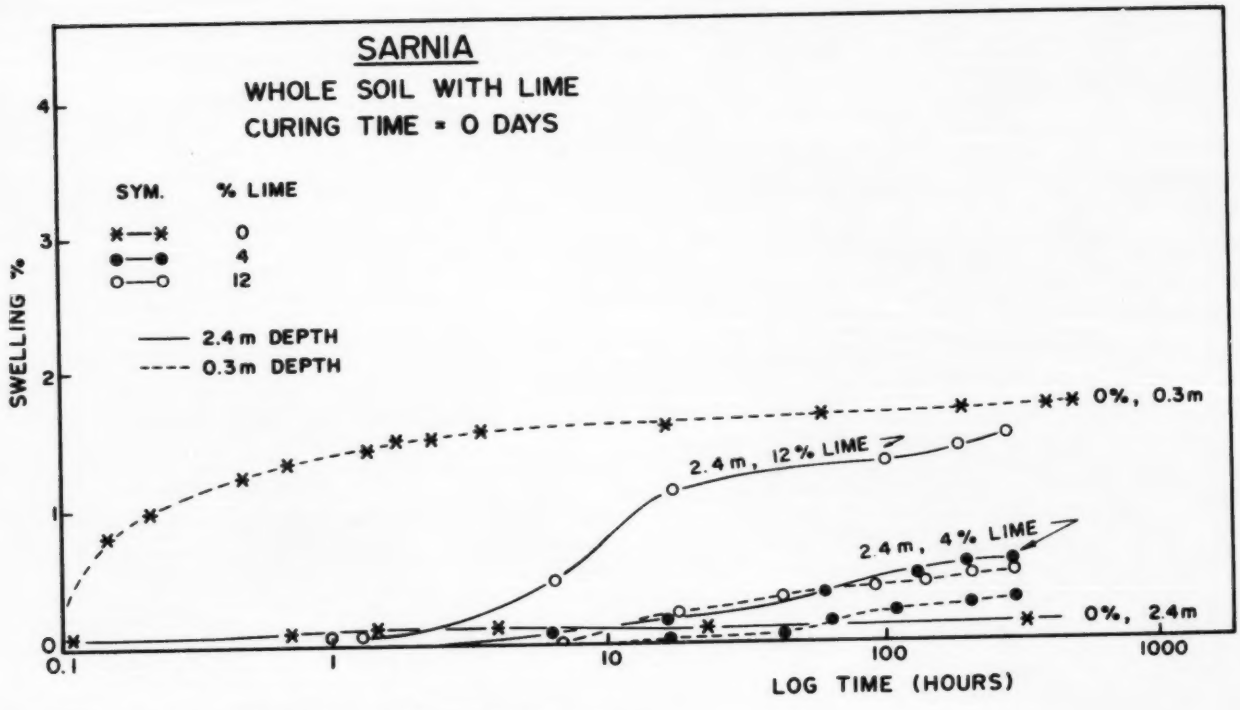

Figure 32/ Swelling vs Log Time of Sarnia Whole Soil with Lime Mixture, 0.3 m and 2.4 m Depth, 0, 4 and 12 % Lime Content

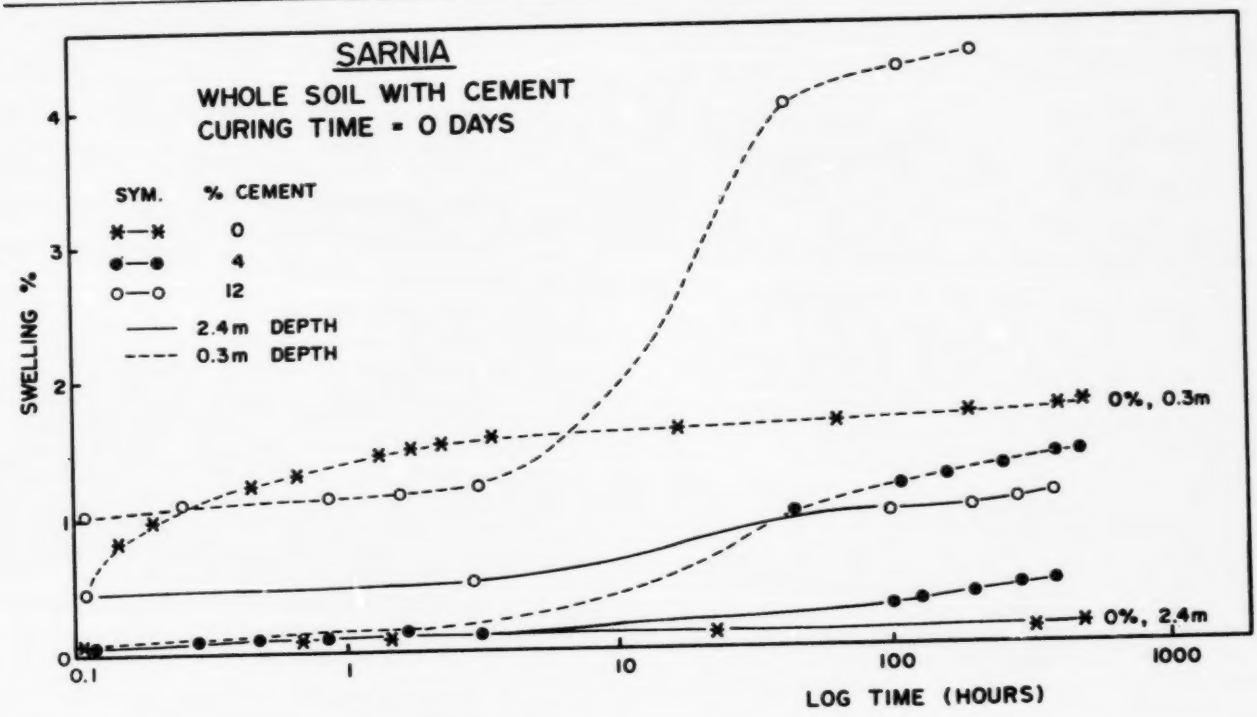

Figure 33/ Swelling vs Log Time of Sarnia Whole Soil with Cement Mixtures, 0.3 m and 2.4 m Depth, 0, 4 and 12 % Cement Contents

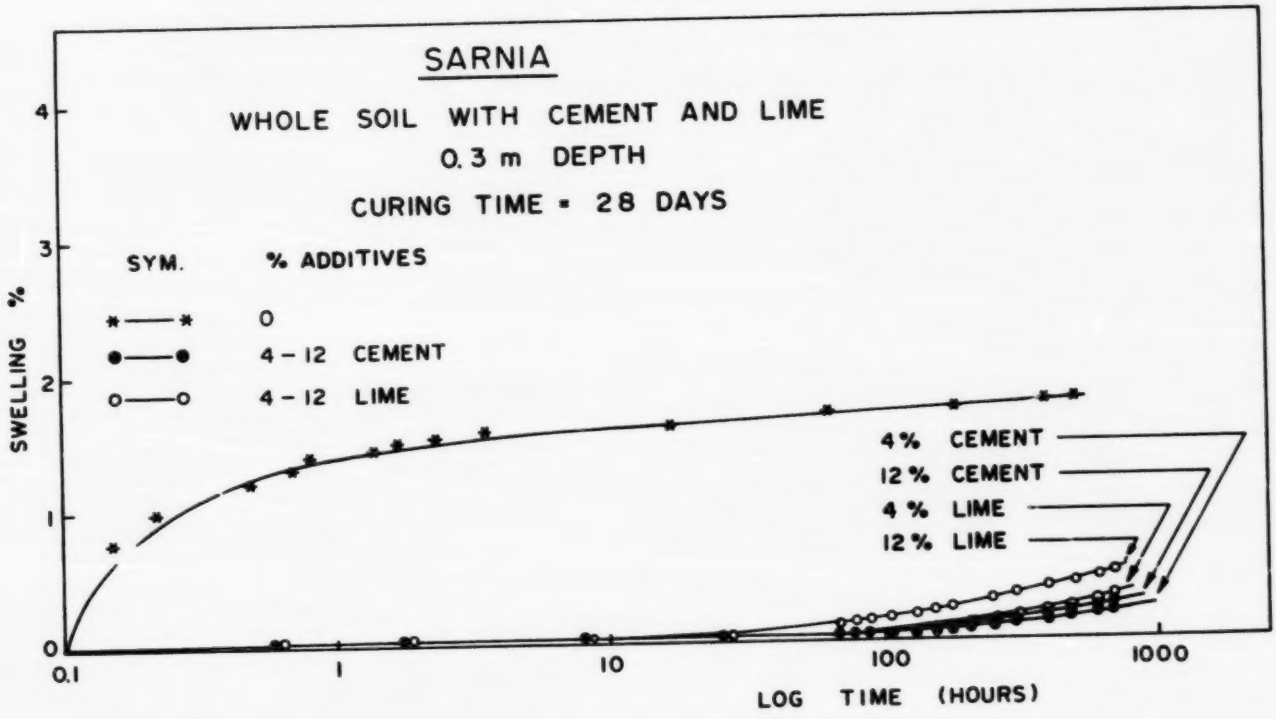

Figure 34/ Swelling vs Log Time of Sarnia Whole Soil with Cement and Lime Mixtures, 0.3 m Depth, 0, 4 and 12 % Additives Content

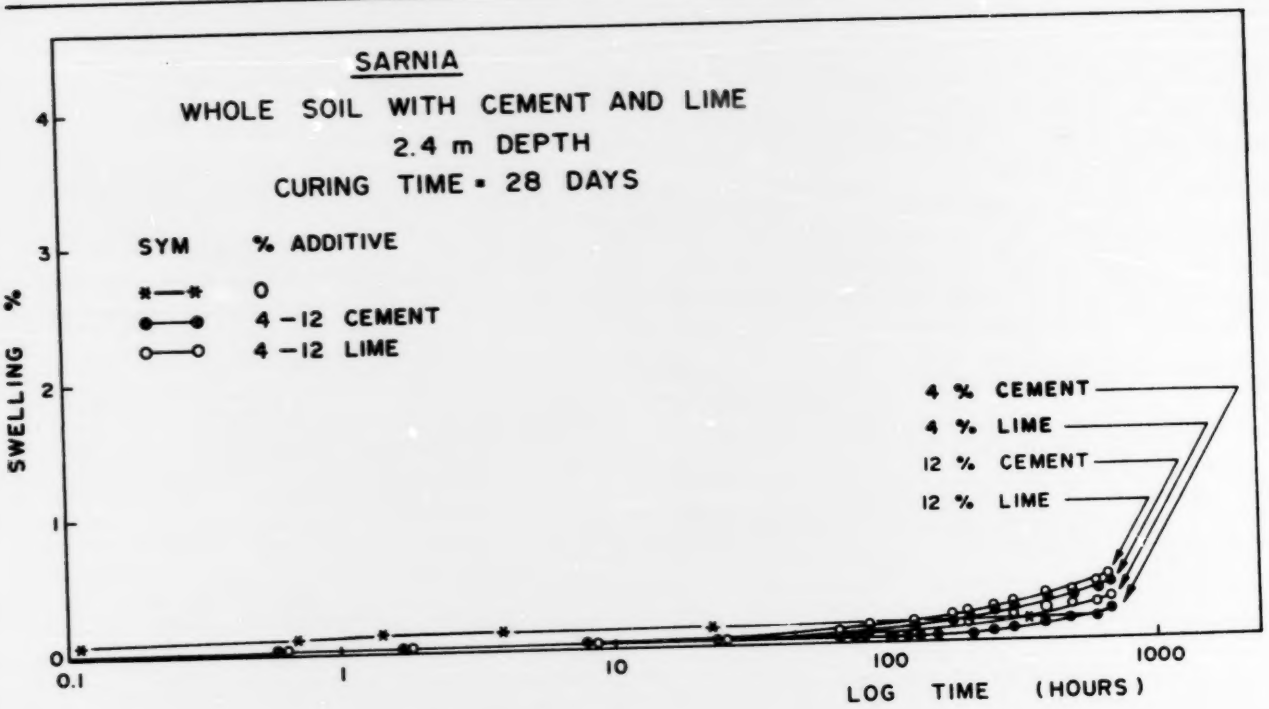

Figure 35/ Swelling vs Log Time of Sarnia Whole Soil with Cement and Lime Mixtures, 2.4 m Depth, 4 and 12 % Additives Content

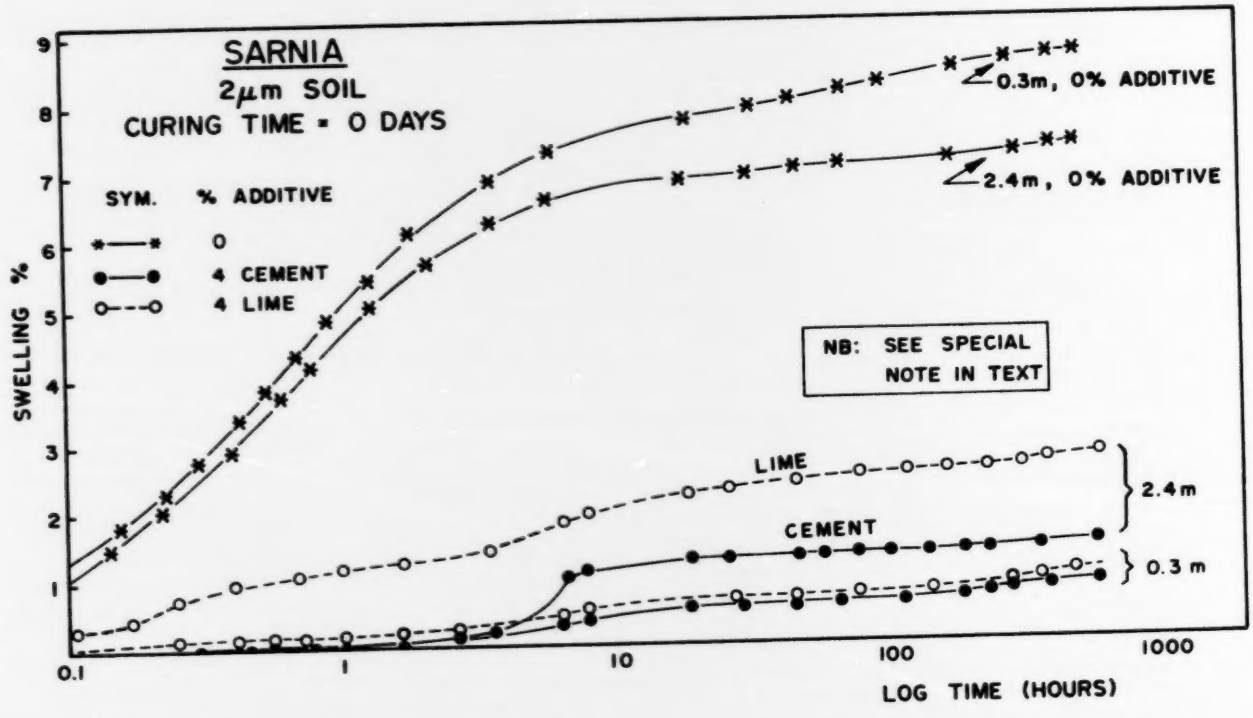

Figure 36/ Swelling vs Log Time of Sarnia <2μm Soil with Cement and Lime, 0.3 m and 2.4 m Depth, 0% and 4% Additive Content

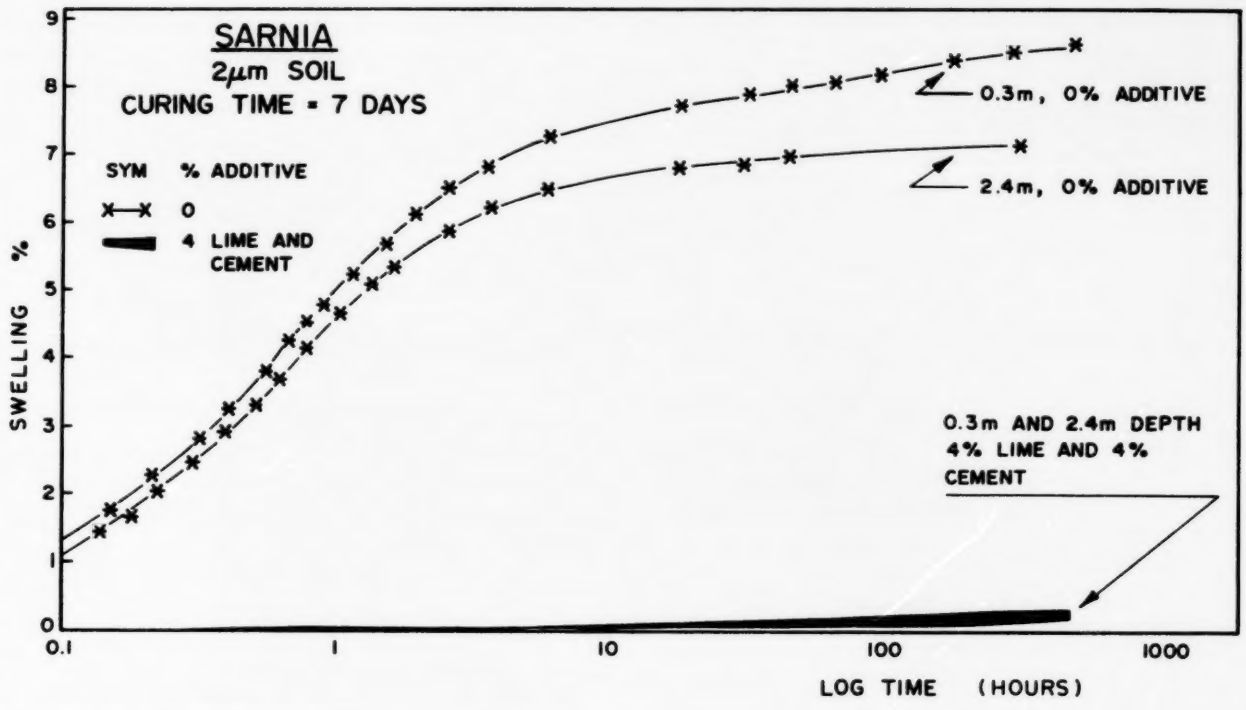

Figure 37/ Swelling vs Log Time of Sarnia 2μm Soil with Cement and Lime, 0.3 m and 2.4 m Depth, 0% and 4% Additive Content

8/ Chemical Test Results and Discussion

For purposes of physicochemical explanation of the strength gain induced by the stabilizers, a series of chemical tests were performed on the test specimens that had been failed in unconfined compression at the end of specific curing times. The chemical information is, therefore, available from time zero up to a maximum of 90 days curing.

The results of the salinity, moisture content and pH measurements are given in Figures 38 to 42 and the X-ray diffraction traces in Figures 43 to 46, inclusive.

8.1/ Moisture Content Measurements

Moisture contents were determined by oven-drying the cured specimens at 105°C for 24 h. The actual moisture content is expressed as W_w/W_s (%) and is believed to represent the percentage of free (uncombined) water in the specimens.

All samples treated with lime or cement show small but significant decreases in water content with time, with the majority of the decreases occurring during the early stages of hydration of the stabilizers (see Figures 38 to 41, inclusive). The curves are somewhat confusing since there was some variation in initial water content so a summary plot of $\Delta\omega$ versus time is presented for 8% additive in Figure 42a. The figure shows clearly that there is greater loss of water with time for the 2.4 m soil compared to the 0.3 m soil. The figure also shows that at either depth, the soil-cement mixes tend to consume more water of hydration with time than the soil-lime mixes. These trends are valid for all percentages of lime and cement.

8.2/ pH Measurement

Measurements of sample pH were carried out on air-dried, disaggregated samples mixed with distilled water in the ratio of 1:2. The samples were shaken for 2 hours, allowed to sit overnight, centrifuged and the pH then run on the supernatant liquid. The pH values obtained are shown in Figures 38 to 41 along with the moisture content values.

Generally speaking, there is a slight increase in pH from about 7.5 to 8.0 as the natural, unstabilized soils are cured. Addition of 2% lime causes an abrupt jump to a pH of about 10.5 or 11 with maximum values of about 12.5 for 12% lime. With time, the samples containing 2 and 4% lime show a slight decline in pH whereas those with 8 and 12% lime retain a constant high pH value for the entire 90-day curing period.

Similar trends are observed for cement treated soils, the pH increasing the values of about 12 for cement contents of 8 and 12%. For 2 and 4% lime, a regular, small decline in pH was observed, however, for 2 and 4% cement, the pH seemed fairly constant in spite of some erratic measurements. Figure 42b illustrates these observations for 2% lime and cement.

8.3/ Salinity Measurements

Salinity measurements were carried out on soil:distilled water mixtures of 1:5 prepared in a manner similar to that already described for the pH measurements.

In Figures 38 and 39, it is clear that addition of 2 and 4% lime increases the salinity of the soil only slightly except for the 2.4 m soil which shows a very rapid decline in salinity between 7 and 28 days of curing. Addition of 8% lime causes a marked increase in salinity for both soil depths. For the 0.3 m soil, the salinity drops rapidly with curing time whereas for the 2.4 m soil, the salinity remains high at an apparent value of 135 g/L. Addition of 12% lime creates a constant high salinity of about 105 g/L for the 0.3 m soil and 160 g/L for the 2.4 m soil.

Addition of cement shows somewhat different trends in salinity. For the 0.3 m soil, the salinity increases with each increase in cement content but reaches only about 60 g/L for 12% cement after 7 days curing compared to 105 g/L for 12% lime. For the 2.4 m soil, the salinity increases with each increase in added cement reaching a maximum apparent value of about 130 g/L for 12% cement compared to about 175 g/L for 12% lime.

In summary, lime causes an increase in measured salinity only if the amount exceeds about 4 to 6%. In contrast, cement causes an increase in measured salinity for all quantities for addition between 2 and 12%.

8.4/ X-ray Diffraction

X-ray powder diffraction patterns for pure hydrated lime, pure hydrated cement, pure soils from 0.3 m and 2.4 m and their mixtures (cured to 90 days) are given in Figures 43 to 46. The uppermost traces for the pure pastes of the lime and cement stabilizers show clearly the many strong X-ray peaks characteristic of their hydrated mineralogy. Similarly, the lowermost traces for the soils show their many characteristic X-ray peaks, appropriately labelled.

The centre traces in each figure are for the soil-lime and soil-cement mixtures cured for 90 days. The most striking feature of each trace is the complete absence of hydration peaks for the stabilizer employed in spite of its 12% abundance. The only obvious changes in the curves with respect to the untreated soils, are the reductions to the size of the clay mineral peaks, especially the smectite peaks.

8.5/ Discussion

Interpretation of physicochemical tests carried out on mixtures of lime and cement with individual clay minerals is fairly difficult in itself. Interpretation of the chemical tests performed on the complex Sarnia test soil must, therefore, be somewhat speculative.

The small moisture content changes with increasing curing time reflect continuing hydration of the lime and cement hydration products with time. In all strength-versus-time plots, strength gain was most rapid early in the curing cycle. The rapid change in $\Delta\omega$ early in the curing cycle (Figure 42a) is, therefore, reasonable and no doubt represents early hydration of the stabilizers.

Increasing lime was observed to cause an increasing pH of the soil-lime mixtures. At high lime contents, the pH remained constant at values in excess of 12 for the entire 90-day curing period, whereas at 2 and 4% lime, the pH declined up to 1 pH unit. Since hydroxyls are released as part of the hydration process, high pH values are generated. With 2 and 4% lime, hydration reactions and interaction with the soil constituents are responsible for the observed decline in pH. At higher lime contents, the high pH is retained and there is probably an excess of lime. The strength tests described previously showed 4% lime to be nearly as effective as 12% lime in developing strength gain. Retention of a high pH in soil-lime mixtures would seem not to be a requirement for successful stabilization.

The increase in pH with increasing cement content is also produced by liberation of hydroxyls during initial hydration. It is not known why the pH did not decrease with time as it did for the lime stabilized soils. Since it is generally believed that a high pH is necessary for successful strengthening of clay-cement mixtures, the absence of a decline in pH may help explain the effectiveness of cement as a stabilizer for the Sarnia soils.

The chemical test results obtained on the Sarnia soils are generally consistent with trends pointed out in the literature by such authors as Shen and Li, 1970 [15], Croft, 1967 [1], and Herzog and Mitchell, 1963 [6].

As a general rule, less than 5% of a mineral constituent in a soil will not yield an X-ray peak on a powder pattern. Since the amounts of cement or lime added varied from 2 to 12% and since the hydration products are probably diffuse and relatively poorly crystallized, an absence of X-ray peaks characteristic of the stabilizers is not surprising. The marked decrease in the strength of the clay mineral peaks is a feature of lime and cement stabilized soils as noted by such authors as Herzog and Mitchell, 1963 [6], Eades and Gim, 1960, [3] and Croft, 1967 [1]. Smectite peaks were generally the most strongly modified as was the case in this study, also the 10 Å Nm illite peaks and the 7 Å Nm iron chlorite peaks were essentially unaffected. The new peak at 1.08 Å Nm between the illite and chlorite peaks that result from lime treatment of the 0.3 m soil (Figure 43) is probably a second order smectite peak associated with partial collapse of an interlayered smectite-chlorite complex.

Finally, work published by Kawamura and Diamond, 1975 [8], contained scanning electron photomicrographs showing webs of needle-like crystals of cement hydrate linking soil particles together in a soil-cement mix. Six scanning electron photomicrographs taken as part of this study are shown in Figure 47. No distinct differences in fabric are apparent to the authors, although the untreated 0.3 m soil does seem to be less aggregated and show more small individual clay minerals than the 2.4 m sample. This observation is similar to comments reported by Ogunbadejo and Quigley, 1974 [11]. Cement is known to act as a dispersing agent as already discussed at some length earlier in the report and the bottom two photomicrographs tend to support this. Both the 0.3 m and 2.4 m samples with 12% cement appear to be less aggregated (i.e., more dispersed) than the compacted natural soils.

Addition of lime to soils from both 0.3 and 2.4 m depth reduced the dry density and increased the optimum water content. The increased aggregation or flocculation implied by these observations is not apparent in the central two photomicrographs in Figure 47.

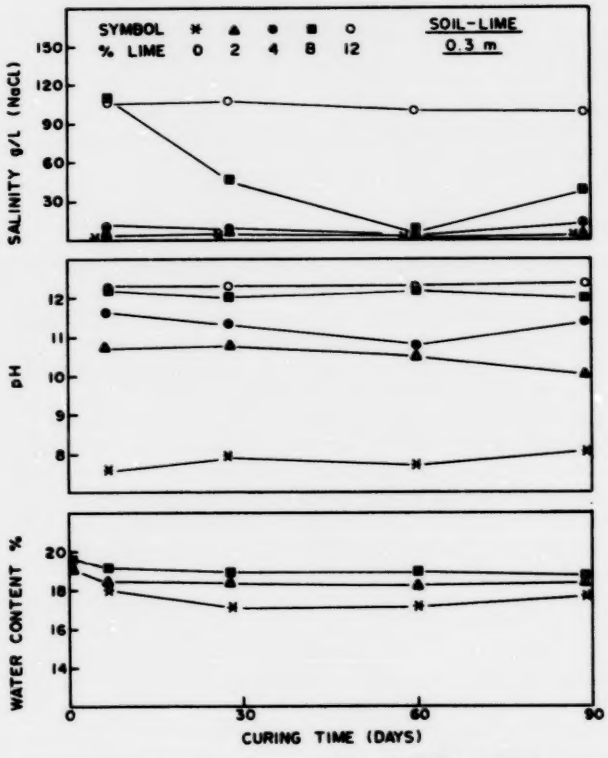

Figure 38/ Water Content, pH and Salinity Changes vs Curing Time, Sarnia Soil-Lime Mixtures, 0.3 m Depth

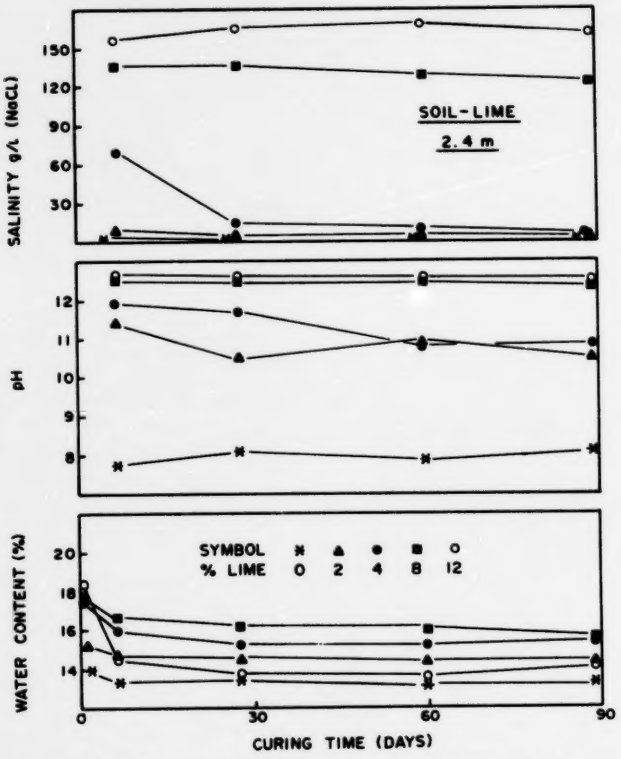

Figure 39/ Water Content, pH, and Salinity Changes vs Curing Time, Sarnia Soil-Lime Mixture, 2.4 m Depth

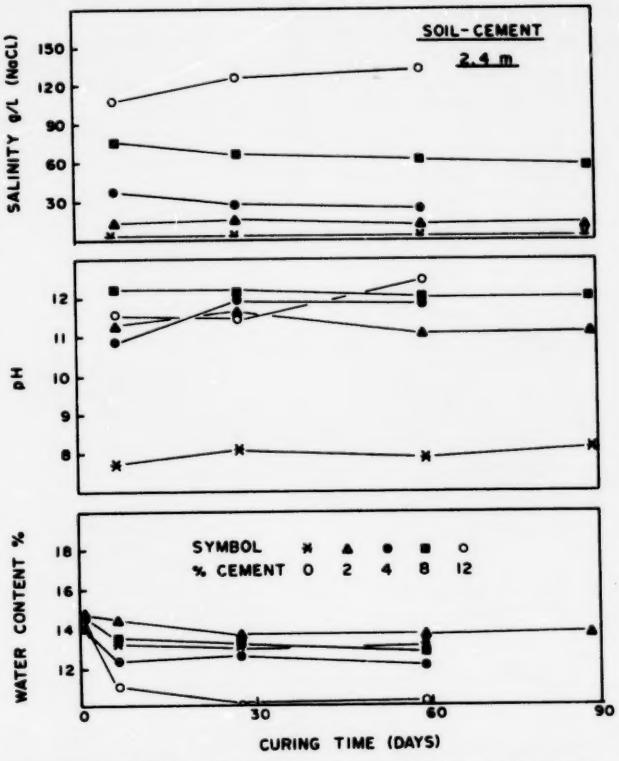

Figure 41/ Water Content, pH, and Salinity Changes vs Curing Time, Sarnia Soil Cement Mixtures, 2.4 m Depth

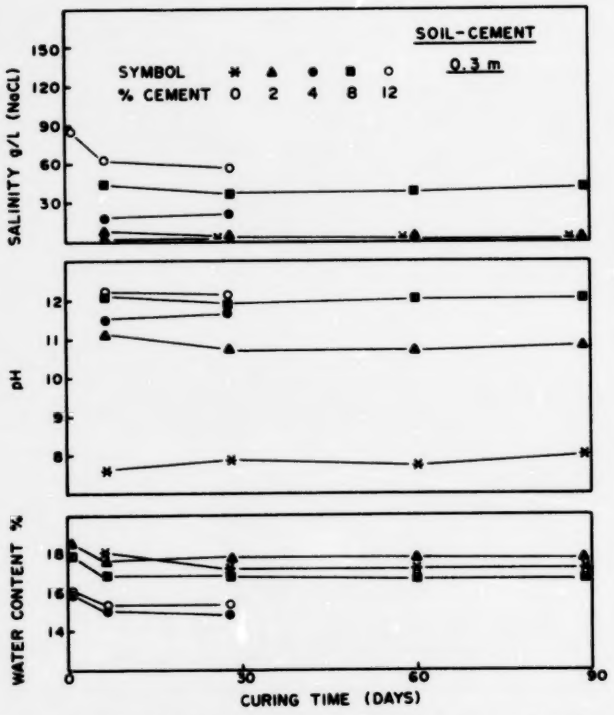

Figure 40/ Water Content, pH, and Salinity vs Curing Time, Sarnia Soil-Cement Mixtures, 0.3 m Depth

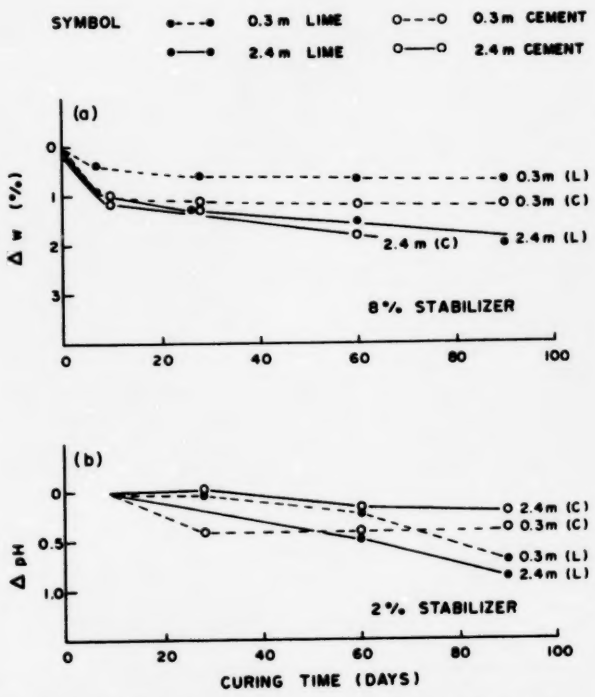

Figure 42/ Selected Changes in Water Content (Fig. a) and Changes in pH (Fig. b) with Curing Time

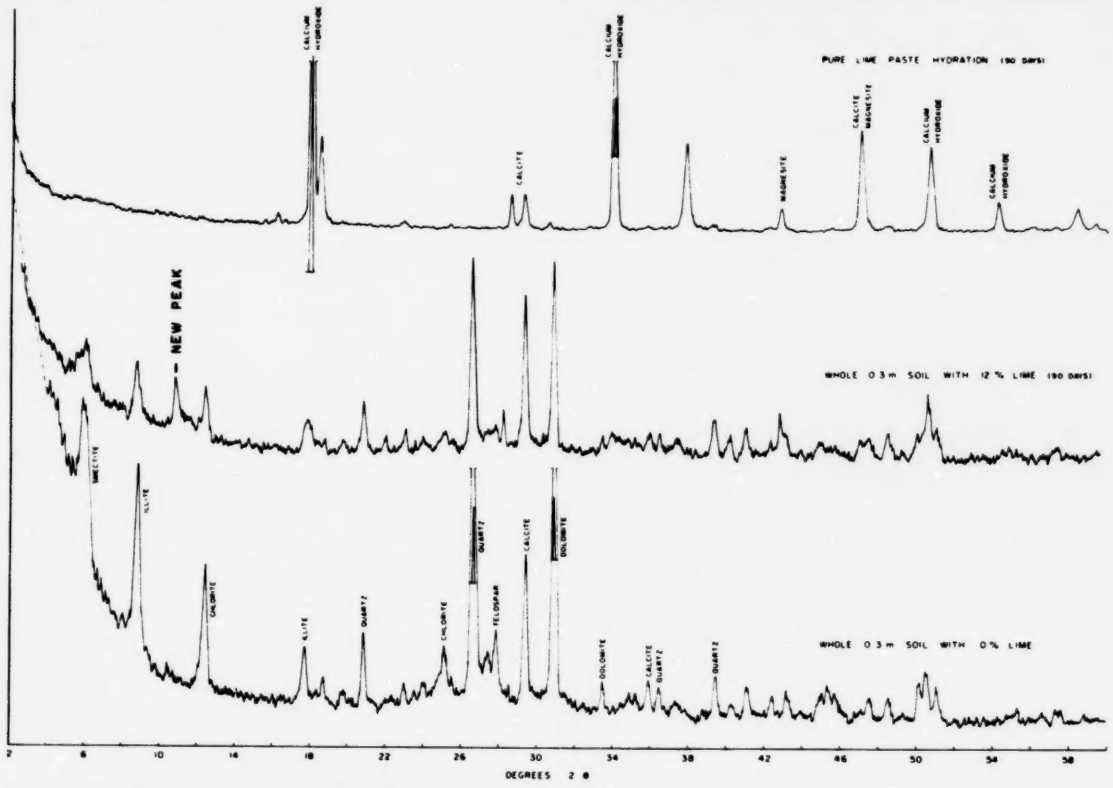

Figure 43/ Comparison of X-Ray Traces of Pure Hydrated Lime with Cured Soil-Lime Mixture and Soil Only, 0.3 m Depth, Sarnia

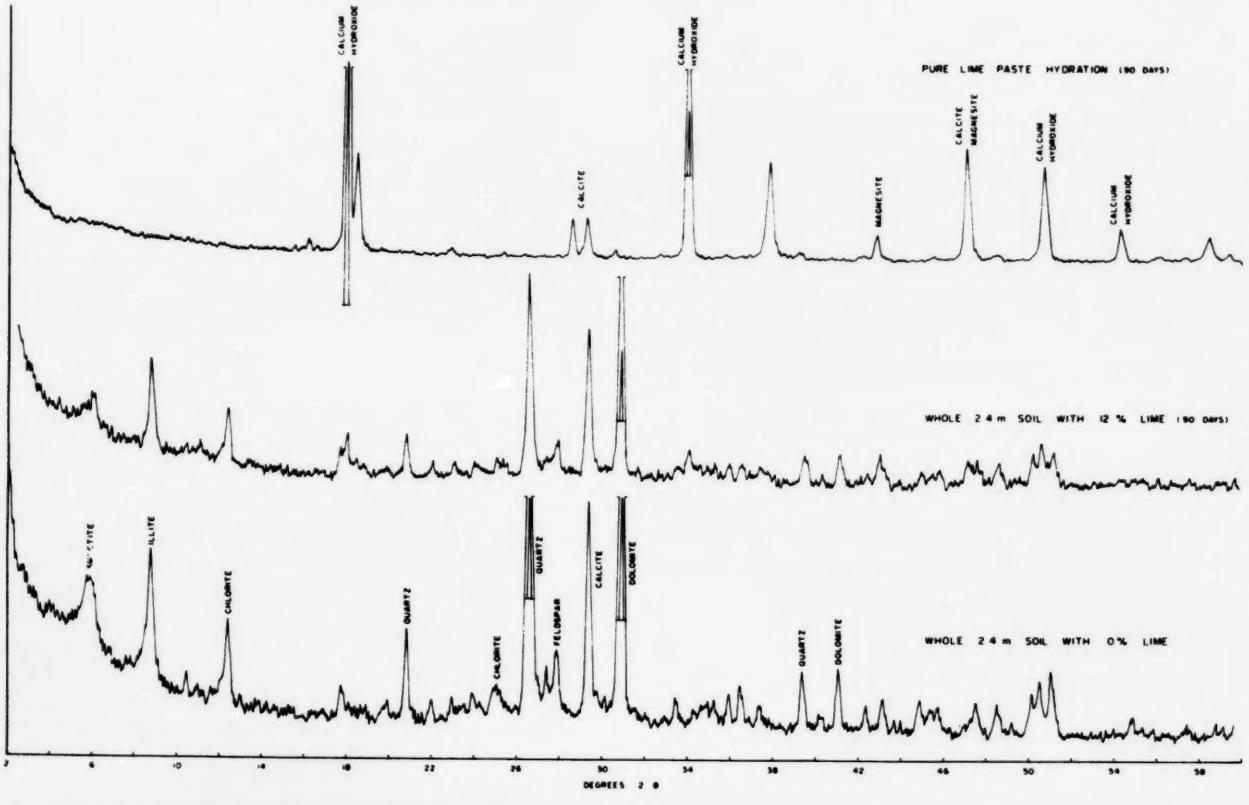

Figure 44/ Comparison of X-Ray Traces of Pure Hydrated Lime with Cured Soil-Lime Mixture and Soil Only, 2.4 m Depth, Sarnia

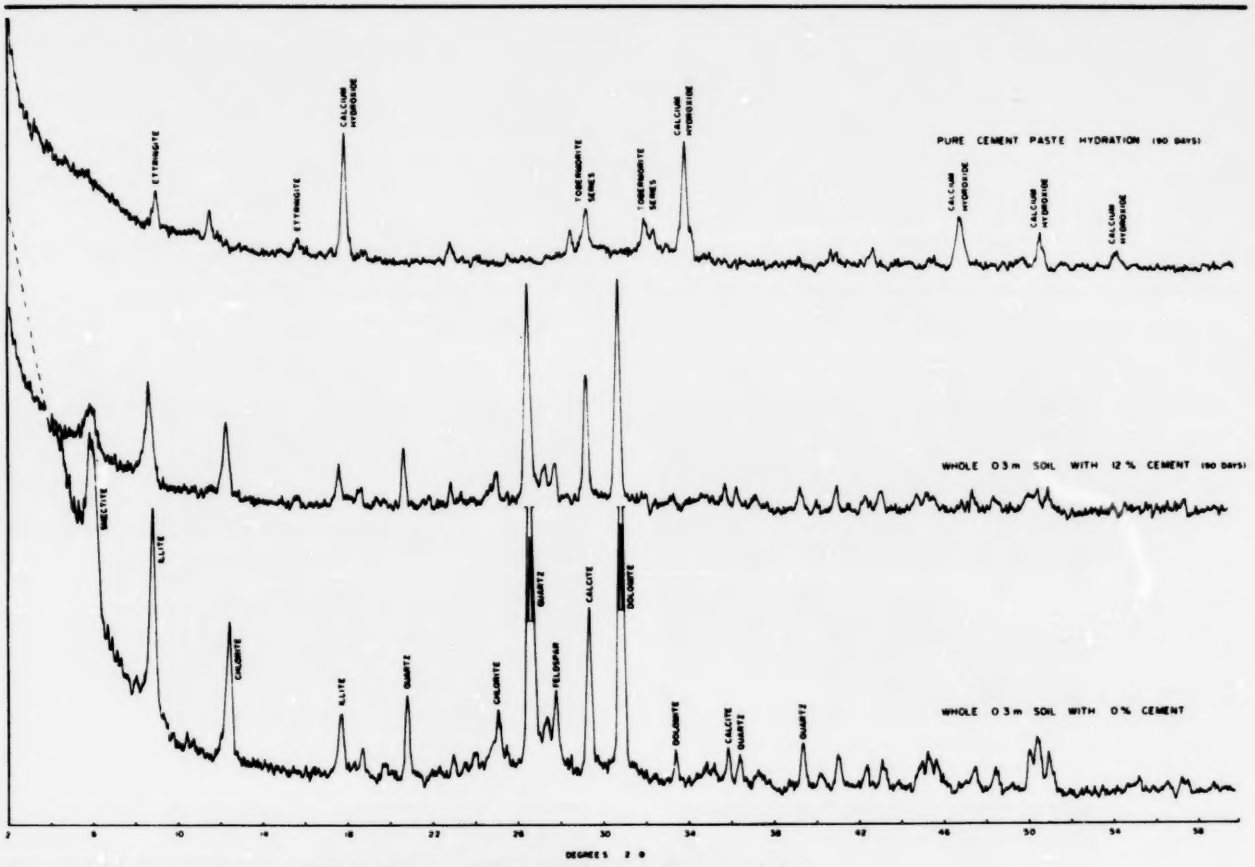

Figure 45/ Comparison of X-Ray Traces of Pure Hydrated Cement with Cured Soil-Cement Mixture and Soil Only, 0.3 m Depth, Sarnia

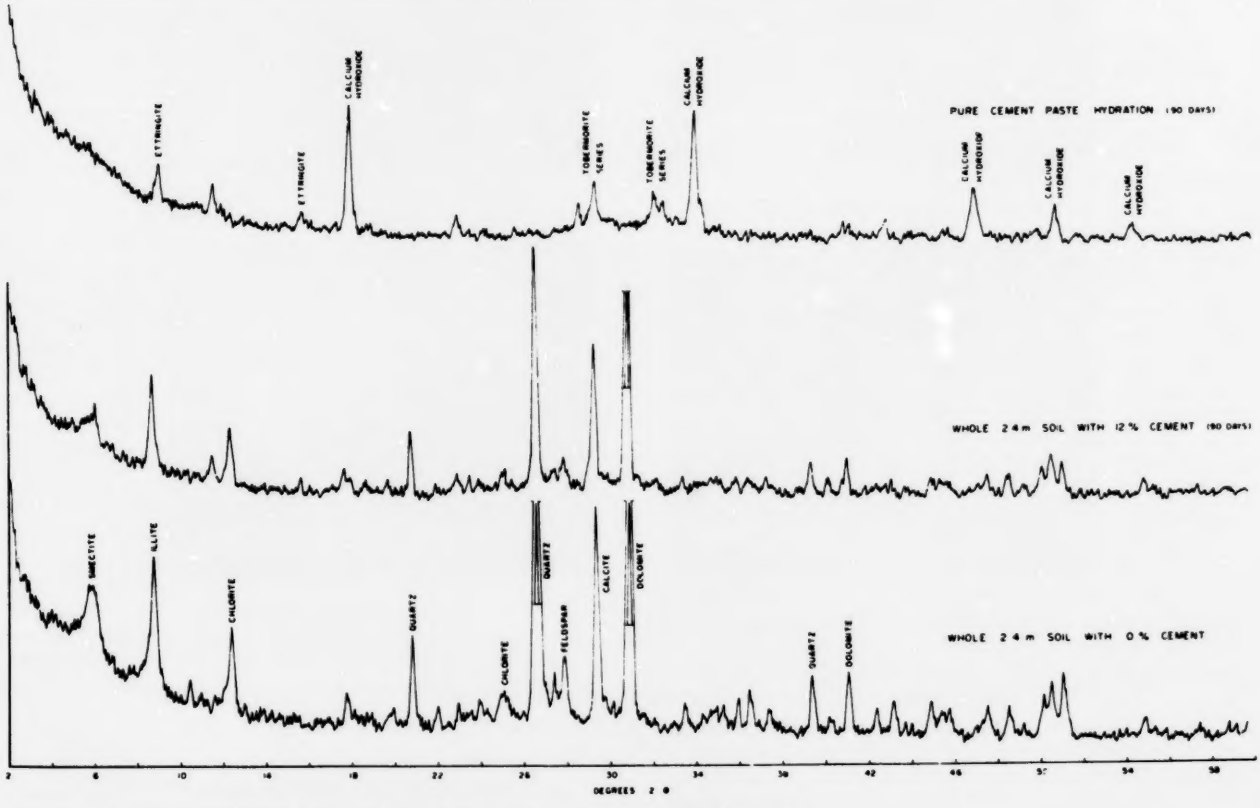

Figure 46/ Comparison of X-Ray Traces of Pure Hydrated Cement with Cured Soil-Cement Mixture and Soil Only, 2.4 m Depth, Sarnia

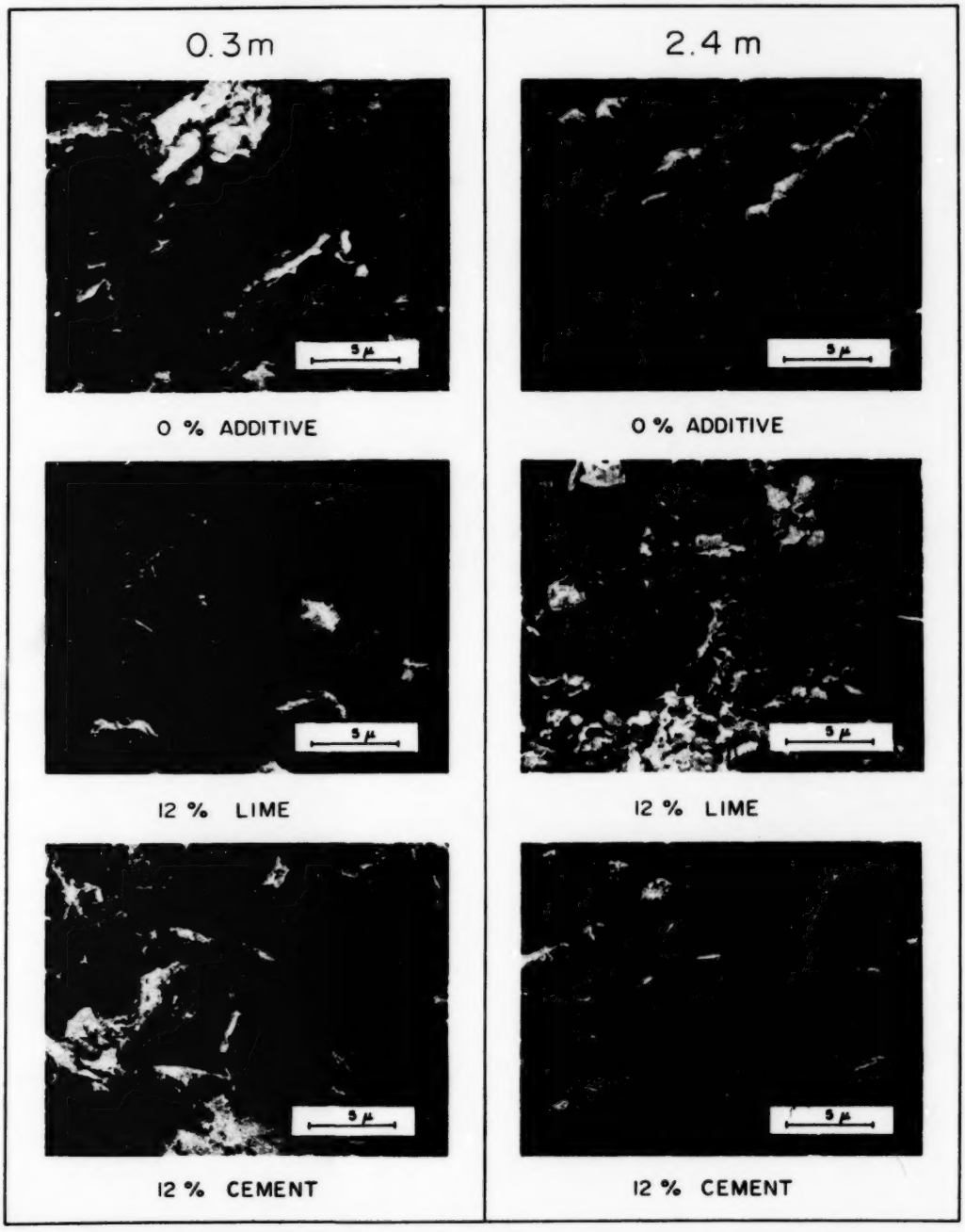

Figure 47/ Comparison of Scanning Electron Photomicrographs of Whole Soil with Whole Soil-Lime or Cement Mixtures; 12% Additives; 0.3 m and 2.4 m Soil Depth, Sarnia; Magnification 5 000 x; 7 Days Curing Time

9/ Special Exploratory Testing

Two sets of special, exploratory tests were carried out in an attempt to understand further the mechanisms of cement and lime stabilization applicable to the Sarnia soils. In one set, the < 2 μm fraction from the 0.3 m soil was treated to extract its amorphous components and then stabilized with 4% lime. In the other set, a sodium silicate solution was used as the moulding liquid to see if it improved the strength of the 0.3 m whole soil stabilized with 4% lime.

9.1/ Amorphous Extraction

The amorphous constituents of soils normally consist of hydrated oxides of aluminum, iron and silicon. They lack sufficient crystal form to yield X-ray diffraction patterns and are generally significantly more soluble than the mineral constituents of the soil. For these reasons, it is normally inferred that they react more easily with the alkaline solutions produced by lime and cement than with the mineral constituents and may be responsible for most of the secondary reaction hydrates, especially in lime stabilization.

In order to check these concepts, the amorphous constituents in the < 2 μm fraction of the 0.3 m soil were removed using the procedures of Segalin, 1968 [14]. The method involves sequential washes with strong NaOH and HCl solutions and is described in detail in Appendix C. The results of the extractions are shown in Figure 48 for Al, Fe and Si as obtained by absorption spectrophotometry. The rate of extraction decreases with each subsequent wash to a constant extraction rate which is assumed to represent dissolution of the mineral constituents in the soil. Extrapolation of the straight line portion of the curves back to the zero wash, yields an approximate value for the amorphous constituents present. As shown in Figure 48, the total quantity of amorphous Al, Fe and Si amounts to about 9%.

Enough < 2 μm soil was extracted at considerable expense to carry out a single strength test and the result compared with previous data obtained on the < 2 μm fraction. The results shown in Figure 49 were completely unexpected. Addition of 4% lime only caused a dramatic increase in the 28-day compressive strength from 1100 kPa unextracted to about 1700 kPa extracted. This increase was originally believed to result from removal of protective amorphous coatings from reactive mineral grains. Subsequent X-ray traces showed, however, that some smectite and chlorite had also been removed by the extractions. Extraction of the smectite may well explain the strength increase.

Use of a saturated sodium silicate solution as the moulding fluid did not improve stabilization of the unextracted clay fraction as expected, except in the early stages of curing. Reaction of the clay mineral constituents of the soil with the lime produced just as effective secondary cementing agents as reaction with the trace additive.

Although Na silicate did not seem to be a particularly effective trace additive when applied to the clay size fraction of the 0.3 m Sarnia soil, it was decided to try it on the whole soil. Na silicate was selected for use in view of its ease of application (in solution in the moulding water) and because studies by others such as Ladd, Moh and Lambe, 1960 [9], indicated that it was one of the most effective additives.

9.2/ Sodium Silicate Additive

A saturated solution of sodium silicate (0.5 molar) was used as the moulding fluid in a series of special tests to assess its effectiveness as a trace element additive. Four percent lime was used as the stabilizer for the whole soil from 0.3 m depth.

The results plotted in Figure 50 show the following:

1/ Addition of 4% lime increases the 28 day compressive strength from about 250 to 500 kPa.

2/ Use of Na silicate as the moulding fluid increases the 28-day strength to 1100 kPa; a rather dramatic increase.

3/ The Na silicate must react immediately with the lime since the strength gain is nearly as significant after 3 days as 28 days.

Although hydrated lime appears to be a successful stabilizer for the Sarnia soils, use of Na silicate as an additive produces an even more significant strength gain. In view of the extraction studies just described, it appears that dissolved silica in the moulding liquid might play a significant practical role in lime stabillzation in spite of the 9% amorphous components present in the surface soils.

It is important to point out that it was necessary to air-dry the soils prior to addition of Na silicate in the moulding water. Otherwise, the amount of added Na silicate would have been insignificant. This air-drying causes permanent collapse of some of the soil smectite and improves its compaction and strength characteristics as discussed by Ogunbadejo and Quigley, 1974 [11]. A single 28-day strength obtained on air-dried soil stabilized with 4% lime is plotted on Figure 50 confirming the significance of air-drying. Nevertheless, it is felt that trace additives of siliceous composition may prove to be effective catalysts in lime stabilization of the Sarnia soils and further research is recommended.

Since silica appears to be the reactive component necessary to achieve improved lime stabilization, finely disseminated siliceous fly ash might prove to be a useful additive and further research in this area is also recommended.

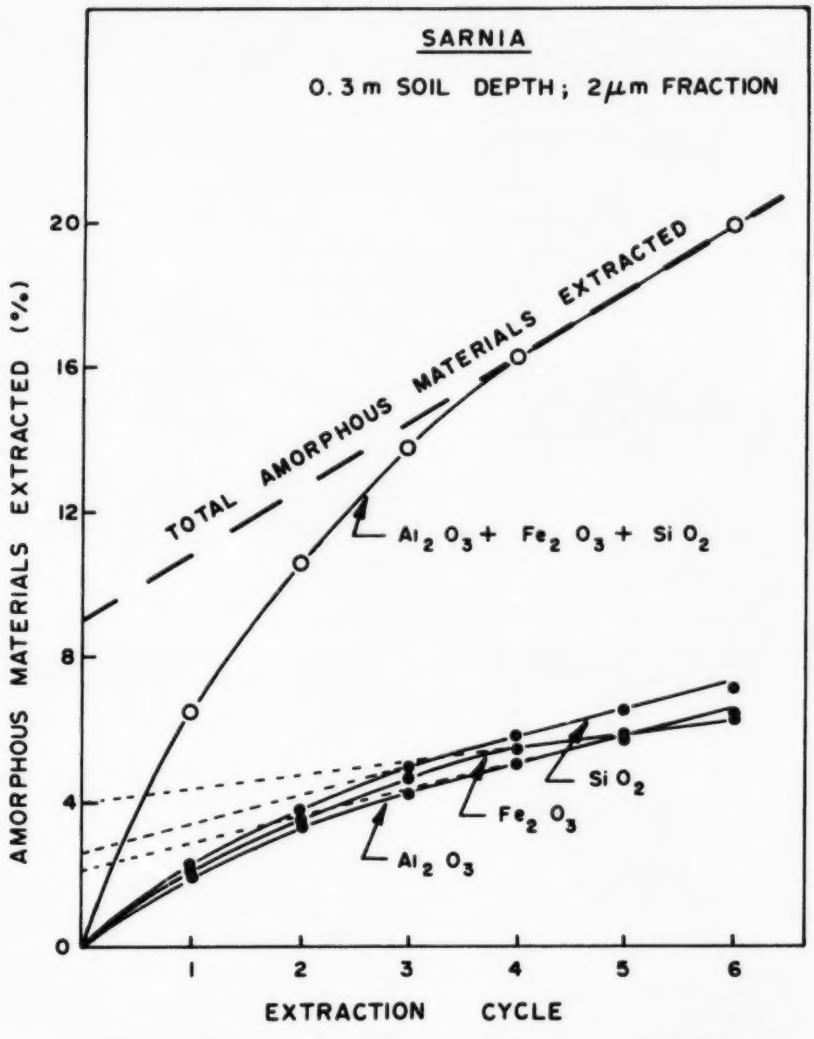

Figure 48/ Amorphous Materials Extraction by Selective Dissolution, HCl/NaOH Treatment, Sarnia Soil

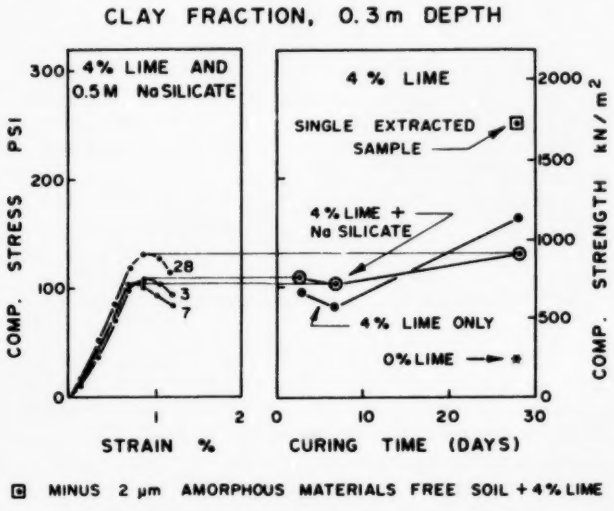
CLAY FRACTION, 0.3 m DEPTH
4% LIME AND 0.5 M Na SILICATE
4 % LIME
SINGLE EXTRACTED SAMPLE
4% LIME + Na SILICATE
4% LIME ONLY
0% LIME
COMP. STRESS PSI
COMP. STRENGTH kN/m2
STRAIN %
CURING TIME (DAYS)
MINUS 2 μm AMORPHOUS MATERIALS FREE SOIL + 4% LIME

Figure 49/ Effect of Amorphous Materials on Strength of Clay Fraction – Lime Mixtures, Sarnia Soil, 0.3 m Depth

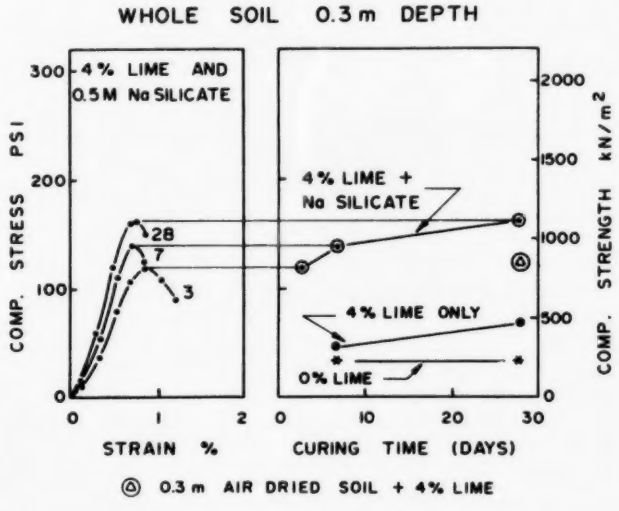
WHOLE SOIL 0.3 m DEPTH
4% LIME AND 0.5 M Na SILICATE
4% LIME + Na SILICATE
4% LIME ONLY
0% LIME
COMP. STRESS PSI
COMP. STRENGTH kN/m2
STRAIN %
CURING TIME (DAYS)
0.3 m AIR DRIED SOIL + 4% LIME

Figure 50/ Effect of Na Silicate on Strength of Soil-Lime Mixtures, Sarnia Soil, 0.3 m Depth

10/ Summary and Conclusions

The effectiveness of hydrated lime and Portland cement for use as stabilizers of clayey soils from Sarnia, Ontario, has been the object of this research project. The research program has included extensive compaction testing, unconfined compression testing and swell testing supplemented by physicochemical control testing. In addition, a small amount of exploratory testing was done on the possible use of Na silicate additives and on the role of the amorphous soil constituents in the stabilization process.

For purposes of clarity, the major conclusions of the current research are subdivided into a series of separate headings as follows:

10.1/ Soil Composition

1/ The test soils from Sarnia, Ontario, are sandy, silty clays consisting, in approximate order, of an abundance of carbonates, illite, chlorite, quartz, feldspar and smectite.

2/ About 9% of amorphous materials are present in the shallow 0.3 m soils as determined by NaOH and HCl sequential extractions.

3/ There is a significant increase in smectite content towards surface from about 3% at 2.4 m depth to about 10% at 0.3 m depth. This increase in swelling clay towards the surface is responsible for the reduced density and higher optimum moisture contents of the surface soils relative to the deeper soils as studied in Project L-3 [12]. Furthermore, the surface soils exhibit swell potentials as high as 6% compared to negligible swell at depth.

10.2/ Lime Stabilization

1/ Hydrated lime is an effective stabilizer of the Sarnia soils. Lime produces a fivefold increase in the strength of the 2.4 m soil after 90 days of curing. It is significant that 4% lime is nearly as effective as 12% lime in strength gain of the deeper soils.

2/ The 0.3 m, smectite-rich soils are more difficult to stabilize. Lime additions of 2 and 4% produced only threefold increases in strength compared to six or sevenfold increases for 8 and 12% lime after 90 days of curing.

3/ Even after 90 days of curing, the soil-lime mixtures appear to be gaining strength, especially the 0.3 m soils.

4/ Lime appears to act as a soil flocculant resulting in decreased dry densities and higher optimum water contents with increased lime.

5/ Addition of lime effectively inhibits swelling of the surface smectite-rich soils provided curing takes place. If distilled water is added to the samples before curing commences, significant swell ($\pm$ 2%) may occur, possibly related to unconfined hydration of the lime itself.

10.3/ Cement Stabilization

1/ Portland cement is a more effective stabilizer of the Sarnia soils than hydrated lime. Cement additions of 8 and 12% to the 2.4 m soil produced seventeenfold increases in strength in 60 days. Even 2% cement produced a sixfold increase.

2/ The 0.3 m, smectite-rich soils are more difficult to stabilize, the strength gain being roughly half that for the deeper soils for additions of 2, 4 and 8% cement.

3/ Unlike the lime stabilized soils, strength gains by the cement treated soils were essentially complete in 30 to 60 days.

4/ Cement acts as a soil dispersant resulting in increased dry densities and lower optimum water contents relative to unstabilized soils.

5/ Addition of cement effectively inhibits swelling of the surface, smectite-rich soils provided curing takes place. If wetted prior to curing, dramatic swelling (up to 5%) occurs with 12% cement. This is probably related to dispersion of the smectite by hydroxyls released from the cement.

10.4/ Stabilization of the Soil Fines

1/ Separation of the < 2 μm soil fractions showed that the clay minerals present in the Sarnia soils are quite suitable for stabilization.

2/ Addition of 4% lime or 4% cement produced approximately equal 60-day strengths indicating significant reaction between the fines and the hydrated lime.

3/ Air-drying the clay-size fraction causes significant permanent collapse of the smectite phase and improved soil characteristics, thus complicating interpretation of the test results.

4/ Even the clay-size fractions of the soils were effectively prevented from swelling by the addition of lime and cement provided curing was allowed. Ultimate swells of only 0.2% were obtained on cured samples with 4% lime or 4% cement compared to swells of 7 to 9% for untreated fines.

10.5/ Chemical and Special Studies

1/ Chemical control tests such as salinity, free water content and pH are useful ways to monitor the progress of curing and stabilization.

2/ X-ray diffraction powder patterns do not show peaks for the reaction products since they do not constitute enough of the sample. Significant changes in the clay mineral peaks were observed as discussed in the text.

3/ The 9% of amorphous constituents appear to play a surprisingly insignificant role in the stabilization process.

4/ Since the amorphous constituents are not a significant source of reaction with added lime and cement, trace additives comprised of silica will probably greatly improve lime stabilization, as shown by one series of special tests.

5/ It is recommended that work be carried on with lime stabilization, especially as influenced by trace additions such as dissolved silica or very finely ground fly ash.

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*Appendix A/ Literature Review

1/ Introduction

The use of cementitious materials such as lime and cement extends back to old civilizations. The Romans, Egyptians and Greeks all used calcined limestone and gypsum in their building construction. A typical mix consisted of broken bricks mixed in a pozzolanic lime mortar which, in the presence of moisture, produced a chemical reaction that resulted in a hardening of the mixture.

Cement, as currently known, was reintroduced about 1790 in England, and 1800 in France, followed by Germany, U.S.A. and Australia. Today, the world's demand for a reliable, scientifically-produced Portland cement is in the order of several hundred million metric tonnes per annum.

Portland cement, due to its chemical and physical characteristics, can be used as a cementitious agent with almost any kind of aggregate. The use of lime as cementitious material, however, is more restricted, mainly due to its inferior strength. Whichever of these two cementitious agents are used in civil engineering, the resultant final solid product must meet a series of specifications such as a certain strength within a given time, a low permeability, adequate durability, and minimum shrinkage, cracking, and surface wear, *et cetera.*

The following factors, which contribute to these ends, should be known and understood:

1/ A knowledge of cement and its properties.

2/ A knowledge of the materials to be used as aggregate.

3/ A reliable estimate of the environmental conditions at the site.

4/ The mix design including mixing time, water:cement:aggregate ratio, etc., plus competent supervision during placement, compaction and curing.

In soil stabilization work, it should be kept in mind that a good cement or lime mixture costs very little more than a poor one, and that their relative performances are vastly different.

Since this project deals with soil stabilization by cement and lime, those factors which yield the "best" stabilized soil are emphasized in the literature review. In particular, clay-cement and clay-lime reactions are emphasized since the Sarnia soils under study consist of abundant illite, chlorite and smectite along with carbonates and silicates.

2/ Portland Cement Chemistry

In the manufacture of Portland cement, clayey materials and calcareous rocks are intimately ground, mixed and subsequently clinkered at 1450°C in a kiln. Gypsum is added during the final grinding to prevent undesirable quick setting.
Four main oxides comprise Portland cement;

CaO	=	60 to 66%
STO_2	=	19 to 25%
$A1_2O_3$	=	3 to 8%
Fe_2O_3	=	1 to 5%

Different types of Portland cement are obtained by changing the relative proportions of the four predominant chemical compounds and by changing the fineness of the cement particles.

The compound compositions of Portland cement are:
tricalcium silicate (alite) C_3S ($3CaO.SiO_2$)
dicalcium silicate (belite or celite) C_2S ($2CaO.SiO_2$)
tricalcium aluminate C_3A ($3CaO.A1_2O_3$)
tetracalcium aluminoferrite C_4AF ($4CaO.A1_2O_3.Fe_2O_3$).

Most of the strength-developing characteristics of Portland cement are controlled by these compounds. The high, early strength is produced by hydration of tricalcium silicate and tricalcium aluminate, and high ultimate strength is produced by dicalcium silicate hydration.

Cements with a wide range of properties are available for particular uses. These properties should comply with relevant standard specifications, which also describe the cement-testing procedure.

To simplify explanation of the hydration mechanisms, the following symbols are employed:
$C = CaO$; $S = SiO_2$; $A = A1_2O_3$; $F = Fe_2O_3$; and $H = H_2O$
Probable hydration mechanisms:

1/ $2\,C_3S + 5H = C_3\,S_2\,H_2 + 3\,CH$
2/ $2\,C_2S + 3H = C_3\,S_2\,H_2 + CH$
3/ $2\,C_3S + 6H = C_3\,S_2\,H_3 + 3Ca(OH)_2$
4/ $2\,C_3S + 4H = C_3\,S_2\,H_3 + Ca(OH)_2$
5/ $2\,C_3S + 6H = C_3\,S_2\,H_3 + 3\,CH$
6/ $2\,C_2S + 4H = C_3\,S_2\,H_3 + CH$

The hydrated components of Portland cement shown above are presented in the most simplified manner possible. The real reactions are very complex and the final products are not easily identified.

The common methods used to determine the hydration products are X-ray diffraction and petrographic microscopy, along with the more recent electron microscopy.

In general, the hydration products of Portland cement have a molecular arrangement similar to a natural garnet, the cement mineral being known as tobermorite which has the approximate composition: $Ca_4(Si_6O_{18}H_2)\,Ca_4,\,H_2O$. The hydration reaction is a function of the water availability and the fineness of the cement particles. The depth of

*Slightly edited version taken directly from concurrent M.E.Sc. thesis of Mr. Luciano DiNardo

penetration of water into a cement particle (ground clinker) and the degree of hydration of the total might be as follows:

Time (days)	Depth (μm)	Hydration (%)
1	10	24
7	47	42
28	78	51

An additional point is that during their hydration, C_2S (belite or celite) and C_3S (alite) produce 18 and 39% of lime hydrate, respectively. If this lime hydrate comes into contact with CO_2 from the air, undesirable carbonates ($CaCO_3$) may form.

3/ Hydrated Lime Chemistry

The use of lime as a cementing agent is much more restricted than Portland cement. Lime was utilized several thousand years ago, but its main use was probably as plaster.

Today, it is used in industry to obtain, process or treat a wide range of material products, as well as for civil engineering activities. The manufacture of lime is relatively simple. It involves the use of certain pure calcareous rocks which are quarried, heated in a kiln at about 700°C or more, powdered, and then usually partially hydrated. The chemical composition and physical treatment may be expressed as follows:

$CaCO_3$ + Heat ⧫ $CaO + CO_2$

$CaO + H_2O$ ⧫ $Ca(OH)_2$ + Heat

If hydrated lime comes in contact with CO_2, the following reaction may occur:

$Ca(OH)_2 + CO_2$ ⧫ $CaCO_3 + H_2O$

As shown previously, the first product obtained on heating is CaO, which is known as quicklime. This can only be stored for short periods of time unless retained in a perfectly anhydrous environment or it will hydrate with atmospheric moisture. To facilitate storage and handling, CaO is partially hydrated, which gives a much more stable product. Further complete hydration plus any reactions between lime and soil or aggregate occurs after it is prepared as part of a mixture to which water is added.

4/ Soil-Cement and Soil-Lime Stabilization

This report is concerned with cement and lime stabilization of two clayey soils located at 0.3 m and 2.4 m depth from the vicinity of Sarnia, Ontario. Soils from these two depths differ markedly in clay mineralogy and engineering behaviour and are discussed thoroughly in the section on Geology and Mineralogy, and the review of Report L-3. Basically, the differences in behaviour are caused by smectite in the surface soils, produced by oxidation of chlorites in the original soil. The following review pertains to stabilization of clayey soils with emphasis on clay-cement and clay-lime reactions.

Current criteria used to assess the suitability of a soil for stabilization are:

1/ The liquid limit must not exceed 50%.
2/ The plastic limit must not exceed 30%.
3/ The maximum percentage of soil particles passing a #200 sieve is 50%.

Soils with higher values than these limits may be stabilized but at extra expense and often with difficulty. The Sarnia test soil under study has index properties and physicochemical characteristics very close to the limits given above.

Various researchers have different explanations for the mechanisms of soil stabilization using cement and lime. These different mechanisms, or reactions, seem to be related in part to the grain size of the soil under consideration. When cement or lime is added to a soil with no fine fraction in it (i.e. sand), all of the strength gain by the mixture is believed to come from hydration of the cementing agent only. In the case of a soil with abundant fines (silty-clay), the strength gained by the mixture is partly due to the hydration of the cementing agent only. In the case of a soil with abundant fines (silt-clay), the strength gained by the mixture is partly due to the hydration of the cementing agent and partly due to the interaction between the agent and the clay particles.

The soil used for this project is composed of a high percentage of clay particles, therefore, a considerable amount of testing was carried out to establish the nature of the interaction between the soil and the stabilizer throughout the complex, time dependent process of strength gain by the mixture. The actual changes in the physicochemical properties of the clay particles after the addition of cement or lime are discussed in the body of the report.

In general, the beneficial effects of lime and cement in soil stabilization have been recognized for many years. A few of the most useful recent references are summarized in the following paragraphs.

Herzog and Mitchell, (1963) [4], and Croft, (1967) [2], determined that the mechanisms of soil stabilization with cement and lime are very similar.

Herzog and Mitchell proposed that the clay particles near to a cement grain will be altered by the dissolution of silica and alumina because of the high pH environment created by the generation of $Ca(OH)_2$ during cement hydration. The dissolved material can combine with calcium ions and form additional cementitious material. These cementitious materials will bond clay particles together in a manner similar to the calcium silicate hydrate bonding from the hydration of cement itself.

Investigation of the reactions of cement-kaolinite and cement-bentonite mixtures revealed that the former mixtures demonstrate a higher strength than

the latter one, and that X-ray traces of cement-bentonite mixtures show a larger breakdown of the clay lattice structure than that observed for the cement-kaolinite mixtures.

Croft, (1967) [2], observed that in cement stabilization of clay minerals, higher strength values were obtained for cement-kaolinite and cement-illite mixtures as compared to cement-montmorillonite and mixed-layer clay mixtures. It was also reported that calcium hydroxide peaks were present on the X-ray traces of kaolinite-cement and illite-cement mixtures, but were absent on traces of mixtures of montmorillonite and mixed-layer clay with cement. Croft concluded that the lime released in the cement hydration process was consumed by the breakdown of the smectite clays much more rapidly than in the case of non-expansive kaolinitic and illitic clays. Croft also speculated that weathered clays such as hydrous illite and vermiculite will behave rather like expansive clays and be more difficult to stabilize.

Kawamura and Diamond, (1975) [6], in their study of stabilization of clay soils against erosion loss, showed the effect of the soil's initial pH on soil-stabilizer interaction. It was concluded that acidic soils will require relatively large amounts of stabilizers (cement or lime) to overcome the effect of soil acidity. The generation of calcium silicate hydrate gel, by direct cement hydration, and by secondary reaction of calcium hydroxide with the silica-bearing clays, acts to bind together individual clay particles and aggregates of particles into a reasonably well knit "reticulated network" providing the stabilizing effect.

Eades and Grim, (1960) [3], investigated lime stabilization of kaolinite, illite, bentonite, halloysite and grundite. Their results showed that increased strength developed in kaolinite-lime mixtures when 2% of lime was added, but for illite, montmorillonite and mixed-layered clays, increased strength developed only after 4 - 6% lime was added. Strength increases are therefore highly dependent on the clay minerals present in a soil. Ladd, Moh and Lambe, (1960) [7], arrived at the same conclusions as Eades and Grim.

Plaster and Noble, (1970) [9], investigated the reactions and strength development of cement-soil mixtures and concluded that clays suffer deterioration after cement treatment from greatest to least in the following order; montmorillonite > kaolinite > illite. They also suggested that the amount of clay in soil-cement specimens appeared to be more important than the type of clay in determining short-term strength development.

Shen and Li, (1970) [10], investigating the stabilization of clay-sand mixtures with lime concluded that the effectiveness of lime-soil stabilization appears to be related to the ratio of the fine grain fraction:lime rather than the percentage of lime with respect to the whole mixture. The pH value of the lime-soil mixture was noted to decrease with both longer curing periods and with larger contents of fine-grain materials. Also, the strength gained by a soil-lime mixture for a given ratio of fines: lime was higher when the coarse-grained soil content was lower.

Neubaver and Thompson, (1972) [8], attributed the gain in stability of uncured, lime-treated fine cohesive soils, to immediate cation exchange, flocculation and agglomeration of soil particles when lime is added. All of these factors will greatly improve any adverse *in situ* soil conditions, expediting construction and increasing subgrade support.

Alexander, Smith and Sherman, (1972) [1], investigated the effect of various limes on clayey soils and concluded that quicklime is generally more effective than hydrated lime for improving the compressive strength of soils. A fineness of hydrated lime of more than 75% passing the #200 sieve does not increase its effectiveness in soil stabilization. In fact, lower densities and less strength may be obtained with the use of extra fine lime. They further recommended a 24-hour loose-curing period for soil treated with quicklime, in order to prevent expansion and pop-outs (loose-curing = curing interval before compaction).

Ladd, Moh and Lambe, (1960 [1], concluded that lime reduces the compacted density of soils but increases their strength after some interval of curing. The use of quicklime in soil stabilization produces an increase of strength of three to ten times within 2 hours of curing. The effect of secondary additives such as sodium compounds improves the strength increase of soil-lime mixtures by several fold, but the effectiveness of the sodium compounds varies with the soil type, additive concentration, and length of curing time.

Other conclusions derived from the previous research are:

1/ The less plastic soils are often less "responsive" to lime than soils with higher plasticity.

2/ To fully develop the maximum strength in clay-lime stabilization, a very long period of time may be required.

3/ Maximum strength in plastic clays usually occurs at optimum moisture content but may occur wet of optimum.

4/ The most effective secondary additive in soil-lime mixtures was found to be sodium silicate.

Kantro, Copeland and Anderson, (1960) [5], discussed the use of X-ray diffraction in the identification of cement hydration minerals; a topic considered elsewhere in this report.

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Appendix B/ Physical Test Procedures

1/ Fractionation (< 2 μm)

In order to establish those controls exerted on the whole soil by the clay mineral fraction, it was necessary to fractionate out the < 2 μm fraction from the whole soil. The sedimentation procedures followed were those of Jackson, 1956. Briefly, the procedure was to disperse a portion of whole soil in an 80 L plastic container and allow it to settle for a certain period of time dependent on the height of the water column, temperature, specific gravity, etc. The < 2 μm fraction still in suspension was then syphoned off and collected by flocculation with dilute $CaCl_2$. Several dispersion cycles were employed to ensure nearly complete removal of the < 2 μm fraction. Typically, a 35 cm deep suspension was allowed to settle for 27.2 h before decantation.

The flocculated < 2 μm material was air-dried and powdered for storage and later compaction testing. A total of seven months of fractionation time was required to collect 11 kg of < 2 μm material from each of the two soil depths (0.3 and 2.4 m).

2/ Unconfined Compression Testing

Unconfined compression testing was carried out using a "Zero-Max" machine to which a calibrated load cell was attached. The strain rate was 0.5%/min and load cell readings were taken at each 0.13 mm of axial deformation. Sufficent points were obtained to plot stress-strain curves for all samples.

All specimens had been statically compacted as described in the section on Procedures in the text of the report. Those soils, mixed with lime and cement were cured as required before compression testing. Two identical specimens were prepared for each specific mixture of soil and stabilizer.

3/ Swell Tests

Swelling tests were carried out on specimens compacted to ρ_{dmax} in a split Harvard Miniature mould from which a central ring of soil 33.3 mm in diameter and 8.4 mm in height could be extracted. After curing for the required period of time, the ring-enclosed disk of soil was placed into the swelling chamber with a porous stone in contact with each face and an applied normal load of 6.895 kPa (1 psi). Distilled water was then added to the chamber and the swelling recorded by dial gauge for about three weeks at 18°C. About thirty swell tests were carried out.

Appendix C/ Mineralogical and Chemical Test Procedures

1/ X-Ray Diffraction

All X-ray traces were obtained on a General Electric XRD-5 diffractometer employing Cu Kα radiation generated at 50 kV and 15 mA. For the oriented clay mineral traces, a scale factor or range of 5000 counts per second (cps) was used compared to 2000 cps for the powder patterns, and 1000 cps for the lime, cement and soil-stabilizer mixtures. All samples were scanned at a rate of 2° 2θ per minute.

Specimens of < 2 μm soil were fractionated from the never dried natural whole soil, centrifuge sedimented onto ceramic plates and X-rayed in the preferred orientation state. The powder traces were obtained on the whole soil (passing #4 mesh) which was ground to minus 200 mesh size. The air-dry soil powder was randomly distributed in glue on a glass plate for X-ray.

The X-ray traces obtained from neat pastes of lime and cement during their hydration process were actually carried out on small blocks moulded at 0.60 and 0.48 water-stabilizer ratios, respectively. The mixtures of soil and stabilizer were X-rayed using specimens compacted and moulded at their optimum moisture contents. All the specimens were X-rayed wet and after known intervals of curing time. During the curing (except for the X-ray time), the specimens (which were wrapped in Saran Wrap) were kept at 20°C in a closed container at high relative humidity and a slight vacuum.

2/ Salinity Measurement

The salinity measurements were done on natural whole soils and soil-stabilizer mixtures.

About 20 g of air-dried natural soil passing #4 sieve was mixed with distilled water at a soil:water ratio of 1:5. The solution was shaken about 30 min, left to stand for 24 h, centrifuged at 2000 rpm and then the clear liquid was tested. The solution resistivity measured is referred to NaCl standards to assess salinity values in g/L. The salinity values recorded for the natural soils are referred to the actual water content of the *in situ* samples.

Salinity determinations on the soil-stabilizer mixtures required extensive physical breakdown until the air-dried mixture passed through a #4 mesh sieve followed by a 2 h shaking period to fully disperse the cemented particles. The sample was then processed as previously described. The salinity values recorded for the soil-stabilizer mixtures are referred to that water content of the mixture obtained by oven-drying at 105°C at the time of testing (i.e., at a specific curing time).

About 150 salinity tests were actually performed.

3/ pH Measurement

The pH measurements were carried out on all the samples on which the salinity was determined. The pH values were measured at 20°C using a pH-meter "Accumet" model 210. The sample preparation procedures were the same as those described for the salinity measurements, the only difference being the soil:water ratio which was 1:2. The pH values were obtained directly from the pH-meter, previous instrument calibration having been done using standard buffer solutions.

About 150 pH measurements were performed.

4/ Extraction of Amorphous Materials from the < 2 μm Fractions

The amorphous materials were extracted from the 0.3 m clay fraction soil following the procedure outlined by Segalin, 1968 [14]. The method is useful for removal of the amorphous Si, Al and Fe components in a soil. It is based on the different dissolution rates between crystalline and amorphous materials. The technique consists of several alternate washings of the soil with 8 N HCl and 0.5 N NaOH. The extracts are decanted separately and analyzed independently for each wash. Each amorphous material extracted during each washing cycle was measured using a Pye Unicam SP191 Atomic Absorption Spectrophotometer. When the rate of extraction decreases and becomes constant, the treatment is terminated. The amount of amorphous material extracted is expressed as a percentage of the whole soil.

This extraction procedure is normally carried out on 0.5 g of soil for purposes of establishing soil composition. In this project, the amorphous materials were determined on 0.5 g of soil and the number of washings or cycles required was established. A 200 g sample, of < 2 μm of soil was then extracted at great expense for use in special compaction tests.

5/ Scanning Electron Microscopy (S.E.M.)

Scanning electron photo-micro photographs were obtained from whole soil samples, natural and stabilized. The soils (with or without stabilizers) were compacted at their optimum moisture contents into small blocks which were then moist cured for 7 days. Then they were air-dried and fractured in tension to give a fresh fracture surface. After mounting on a glass slide, the viewing surfaces were air-blown to remove loose particles, and sputter coated with a layer of gold-palladium. Later, S.E.M. photographs were taken by technical staff in the Dentistry Faculty using the Mark II scanning electron microsope housed there.

About 10 specimens were prepared and scanned.